Analysis of Existing High-Rise Reinforced Concrete Structures in Malaysia Subjected to Earthquake and Wind Loadings

by

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CERTIFICATION OF APPROVAL

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Approved by,

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(Assoc. Prof. Dr. Narayanan Sambu Potty)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

September 2012

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons

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MOHD REDZUAN BIN ABD HAMID

ABSTRACT

The purpose of this study was to investigate the effect of earthquake and wind loading on high rise structure in Malaysia. Conservatively, structural design in Malaysia overlook the significance of both loading (earthquake and hurricane) as they rarely take place in this region. However, occurrences of several tremors in neighbouring countries were enough to put us in fear. So, it is the time for us to revise existing structures to check for their reliability in facing any unforeseen natural disaster. This paper will be the key for any enhancement necessary to be implemented to our existing structures. The method of study mainly involves extended analysis of high rise frame structure and its behaviour towards movement and shakes in complying with UBC 1997 and IS 1893. Starting with simple vertical load analysis and then imposing earthquake and wind loading, the integrity of the frame structure is analysed. The behaviour in term of displacement, and the serviceability limit state of a particular structural will be studied and evaluated in order to quantify the maximum magnitude of lateral loads whereby a rigid frame structure could withstand before it starting to fail. This study analytically proves the outstanding performance of gravity designed structure towards typical wind and seismic conditions in Malaysia (35m/s for wind speed and 0.03g for seismic). However, existing structures in Malaysia without lateral loads design are expected to fail whenever wind and seismic forces are going beyond the typical conditions. The whole analysis demonstrates the understanding of certain code of practices, establish the ability of analysing and deducing the behaviour of structures, handling existing software, and interpreting the results and data to provide relevant comments and modifications whenever needed.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Background of Study

Recently, Malaysia has experienced a series of tremors that put us in fear. Even though the real earthquake only happened a few hundred kilometres away from our country, but the effects were truly significant especially towards high rise buildings in urban area. For example, Persanda Apartment in Shah Alam was shaking tremendously in April 11, 2012; affected by earthquake of 8.9 Richter scale which happened in Acheh, Indonesia. The tremors were significant enough to call all hundreds of occupants out of their home in panic and havoc. The same scenario happened in certain areas in Malaysia especially in west-coast areas. Due to these circumstances, we might begin to question whether our structure can survive in any worse geological nightmares and natural disasters. For several decades, Malaysians believed that our region is immune to any active geological activities, but we may be wrong considering recent geological trend manifested in neighbouring countries. Other natural disasters like hurricanes and typhoons also should not be disregarded as these disasters are totally out of control by any human power. So, it is very important to be ready and fully prepared to face any worst case scenario especially in term of providing the safest yet most reliable structures and buildings for human shelters and protection. Having said that, to be one step ahead, especially in dealing with uncontrollable natural disaster, it is important to be always aware with the after-effects of previous documented disaster and analysing it for the good of future improvement and enhancement whenever needed for each and every aspect of it.

1.2 Problem Statement and Importance of Study

After viewing this issue thoroughly, we may question ourselves whether Malaysia is ready to face another unpredictable series of tremors that may occur in higher magnitude and scale? If we just look this issue on surface, we may not see any casualty and permanent damage involving this series of tremors yet, but do we always have to be optimistic all the time without doing anything about it? Luckily, most of the tremors so far occurred in low populated area with limited high rise structures, but are we ready to face the same magnitude of panic and havoc in higher populated area with lots of superstructures and high rise building like in Kuala Lumpur? The real question is, how much do we consider the integrity of our structures in order to withstand any magnitude of tremors that may occur at any time and any place without any particular warning?

So, this study is going to be extremely important in order to answer all the above questions clearly by analysing our existing high rise structure in term of their structural integrity in facing earthquake and hurricane. We are going to see how far our structure can survive in various shaking conditions as well as any possible of storm and hurricane in order to rationalise any enhancement and reinforcement required to upgrade our structures that was conservatively designed without any account on these types of natural disasters.

1.3 Objectives

The objectives of study are as follows:

(i) To investigate the behaviour of structures subjected to earthquake and wind loadings

(ii) To identify load condition under which structures are unsafe (wind and earthquake load) in complying with different codes.

(iii) To analyse the integrity of structures in facing future earthquakes and storms

1.4 Scope of Study

This study focus on the behaviour of high-rise reinforced concrete frame structures as designed by gravity loads only to the additionally imposed wind and earthquake loads. Yet there is no visible and physical structural failure due to these loads in Malaysia, the analysis will observe the displacement (serviceability limit states) of structural members in order to interpret the possible failure formation of the structures. Simulation of various magnitude of wind load will be imposed to different height of structure to study its significance and influence towards the structural integrity. Meanwhile, earthquake loading analysis will determine the approximate magnitude of ground acceleration where failure of the structures may happen. Once the structural failure configuration determined and understood, possible enhancement methods and improvement of structural members will be recommended.

1.5 Relevancy and Feasibility of Project

Realising the possibility of unpredictable natural disaster to occur in our region, taking account for both wind and earthquake loading for the design of new structures may need to be considered by all structural engineers in Malaysia. However, more concern is focused on pre-existing structures that hold unknown level of structural integrity in facing this disaster. So, thorough research and study need to be done as soon as possible. As it is mainly involving structural analysis using software, this study will not be cost-consuming. The study may involve some simulations, analysis and prediction on modelled structures as subjected to lateral loads. 29 weeks of allocated time frame for both FYP 1 plus FYP 2 is considered enough to perform this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Literature Review is an essential part of a study to clarify underlining processes or component analysis of a research topic. For this study, the following literature review will establish a clear tie between the works that are going to be done in this research with previous findings and analysis. There are seven (7) sub-sections in this chapter that are going to enlighten the readers regarding the study. First sub-section titled ‘History of Earthquake in Malaysia’ generally tells the readers about series of earthquake in Malaysia and its severances while the next part, ‘Response of Buildings to Earthquake Loading’ will give the ideas of previous documented failure of structures and their behaviour to this loading. Later, ‘Design Practise in Malaysia’ will elaborate on the development of seismic study and awareness in our country. ‘Modelling Using Excel Spreadsheet Program and STAAD.Pro’ will discuss on tools available to be used in analysing structural behaviour. There are two sub-sections of Method of Analysis, titled ‘Moment Distribution Method’ and ‘Bending Moment Diagram’ which going to elaborate on steps and procedures of analysis as well as the interpretation of them. Last but not least, ‘Land Optimization and Buildings in Kuala Lumpur’ mainly going to give the quantitative ideas on development trend in Kuala Lumpur.

2.2 History of Earthquake in Malaysia

Malaysia has experienced several significant tremors triggered by earthquake in neighbouring countries like Indonesia. Compared with decades ago, where earthquake or tremors seems to be impossible to happen in our region, latest trend showed that more tremors detected beginning the year of 1984 with the strongest magnitude of 5 to the Richter scale in Kenyir Dam area. Some series of tremors happened in Bukit Tinggi, Pahang on Nov 30, 2007, followed in Jerantut, Pahang on March 27, 2009 with magnitude of 2.6 Richter scale and most significantly in Manjung, Perak on April 29, 2009 with the magnitude as high as 3.2 Richter Scale (Loh & Bedi, 2009).

In Sept 2009, thousands of people in KualaLumpur were affected by a strong earthquake with a magnitude of 7.6 on the coast of Sumatra in Indonesia. The epicentre of the earthquake was recorded 80km deep, and 475km south-south-west of Kuala Lumpur. Occupants from 28 storey of Wisma IMC at Jalan Sultan Ismail and other tall buildings were evacuated. At the same time, resident of high rise condominium in Bangsar, also felt the same tremors giving us a significant warning that we should not ignore the threat from this disaster no more (Spykerman, 2009).

Although it is understood that Peninsular Malaysia is situated on a steady part of the Eurasian Plate, structures especially those built on soft soil are occasionally exposed to tremors due to far-field effects of earthquake in Sumatra. It was proven that for the last few years, tremors were felt in tall buildings in Kuala Lumpur (Balendra & Li, 2008). The tremors were actually caused by 1500 km long Sumatra Fault system in which only 350 km away at the closest point from our Peninsular Malaysia (Brownjohn & Pan, 2001).

On Nov 26, 2004, Malaysians were extremely in shocked by Sumatra-Andaman earthquake that brought the most significant and direct impact towards our country with the magnitude of 9.15 Richter scale. The tsunami produced much numbers of casualties. In Malaysia alone, 50 deaths were recorded and havoc was tremendously felt for people in high-rise building in western states of Peninsula (Koong & Won, 2005)

Massive earthquake in the Sumatran zone tends to generate a very long extent of ground motion in about 300s with major period between 1.5 and 2.7s which happened to be very close with the natural periods of medium and high rise buildings in Malaysia. On the other hand, potential moderate local earthquake of magnitude up to 5.8 also may generated by some active faults in Sabah. (Abas, 2001)

So, it was proven that Malaysia is no longer immune to earthquake disaster as many people believed before. However, the most frightening fact behind this issue is that there was no existing structure except KLCC and Penang Bridge was designed to resist the earthquake forces. Conservatively, structures in Malaysia were designed based on vertical dead load and live load without taking allowance for side to side load caused by earthquakes. As the structural integrity may be compromised by significant tremors, deeper studies and assessments required to be done especially involving high populated buildings like hospital, school and office in order to take any relevant pro-active steps to manage this issue. (Bakhari, 2009)

2.3 Response of Buildings to Earthquake Loading

Basically, both wind and earthquake loads are applied horizontally on the buildings. However, there is major distinction between them in relation with their destructive way. Wind loads damage a building externally by their direct pressure while earthquake loads tend to generate inertial forces that damage the buildings internally. The resistance of building towards these loads are dependent on their mass, size as well as their configuration. (Har & Golabi, 2005)

As the ground started to shake vigorously due to earthquake, the structure will tremble and inertial forces produced internally in the structure to resist the sudden movement. Horizontal shear force then will be imbalanced and displaced causing the structure to be weakened and compromised. During this condition, any additional vertical loads will directly causing the structure to damage and collapse (Ambraseys, 1988). Realising this situation, Koong and Won, (both are Operation Directors of Sepakat Setia Perunding Sdn. Bhd) came out with several important outlines and design principles in order to minimise damage due to earthquake.

Firstly, they believed that total mass of the structure need to be minimised, taking account of Newton 2nd Law which stated that force is equal to mass multiplied by acceleration. Considered earth acceleration and movement would be constant, and then the internal force of smaller mass structure would be lower compared to higher mass structure. Another important principle is that the structure should be simple, symmetric and regular in plan and elevation. This principle actually concern regarding the centre of mass of the whole structure where if it happen to be the same with the plan's geometric centre, then unnecessary rotation could be prevented. Besides that, mass, stiffness, strength and ductility also need to be distributed uniformly to prevent any soft stories where all stories would share equal demand of seismic load thus increasing earthquake resistance capacity. (Koong & Won, 2005)

The basic design philosophy of earthquake resistance structures is that they must be fully operational within a short time after a minor shaking. The repair costs also expected to be small. On the other hand, after moderate shaking, the building should be able to operate as the repair and strengthening of the damaged main members is finished. Then, after strong earthquake, the building is expected to be able to stand for people to be evacuated and property recovered as the building may become dysfunctional for further use. (Murty, 2004)

During earthquake, waves are generated that may be slow and long, or short and rapid. *Period* is the length of full cycle in seconds while the inverse of it is called *frequency*. Technically, every matter, including buildings have their own natural or *fundamental period*, at which they will start to vibrate as being shocked abruptly. Theoretically, whenever the natural period of the building corresponded with the period of the shock wave, the building will resonate, and the vibration shall amplify several times. The fundamental period is proportional to the height of a building. (Lorant, 2012)

Figure 2.1 Variation of Fundamental Period with Height

In some conditions where dynamic amplification occurred, the building acceleration can be doubled or more of the ground acceleration at the base of the building. Generally, buildings that possess shorter fundamental period will suffer higher accelerations but smaller displacements. On the other hand, longer fundamental period buildings tend to experience lower accelerations but larger displacements. (MCEER, 2010)

Considering two most popular construction materials of high rise buildings; reinforced concrete and metal, it is understood that when it comes to earthquake resistance, metal reacts better compared to concrete. This is due to its flexible properties that allow the structures to sway with the movement of the earthquake without breaking. Concrete on the other hand possess the unyielding nature that may cause crack to the buildings during earthquake. (Nutt, 2007)

Nowadays, more concern is given to the study of vulnerability of existing reinforced concrete structure, designed for gravity only to seismic loading (Magenes & Pampanin, 2004). Figure 2.1 shows failures of concrete buildings due to seismic loading.

a) b) c)

Figure 2.2 Observed effects of interaction between infills and bare frame: a) shear failure of column and b) exterior joint shear damage (Bonefro, Molise 2002); c) global collapse for soft storey mechanism (Izmit, 1999, NISEE image collection)

Technically, gravity load represents vertical loadings due to dead loads and live loads of a structure. Dead loads including the self-weight of the concrete while live loads are taken account from code of practice. On the other hand, analysing a reinforced concrete structure to respond to earthquake and wind load, the static lateral loads of both loading are first to be analysed (Adnan & Suradi, 2008).

Generally, structural damage involves the failure of yielding of structural members. Those members that support floors and roofs as well as restraining the structure from lateral loads such as shear walls and bracing frames are considered as primary structures. Failure of primary structures can lead to collapse. On the other hand, partitions, stairs, windows frame that are categorised as secondary structures may suffer damages without compromising the integrity of the building. Failure may be ductile or brittle. Usually, brickwork elements and concrete exhibit brittle failure while steels fail from their ductility (IPENZ, 2011).

One of the critical parameter in evaluating the performance of a high rise structure toward lateral load is by observing the deformation of the structure whereby it will determine level of damage based on the degree of deformation in components and system. Deformation can be further categorised into three (3) types; overall building moments, story drifts, and inelastic deformations of structural components and elements. (Willford, Whittaker, & Klemencic, 2008)

In order to analyse the performance of a structure in resisting earthquake, it is important to consider 2 important requirements and guidelines in structural design code, namely Ultimate Limit State and Serviceability Limit State. For amenity retention (Serviceability Limit State), the building should respond elastically. Though minor damages may be inevitable, but the building must still be fully operational. However, damages should be preserved and control to avoid any structural failure to happen. For collapse avoidance (Ultimate Limit State), the level of risk involving life safety is taken into account in acceptable low allowance. The main concern is to prevent the building from collapse. The damage may or may not be structural and the repair may not be economical (Paulay & M.J.N, 1992)

Most of the time, high rise buildings are likely to behave as a laterally loaded vertical cantilever. Inertia forces generated by earthquake usually considered acting as lump masses at every floor. The magnitudes of forces are considered to be the product of seismic mass (dead load plus long-term live load) at each level. The loading pattern is showed in figure below (King, n.d)

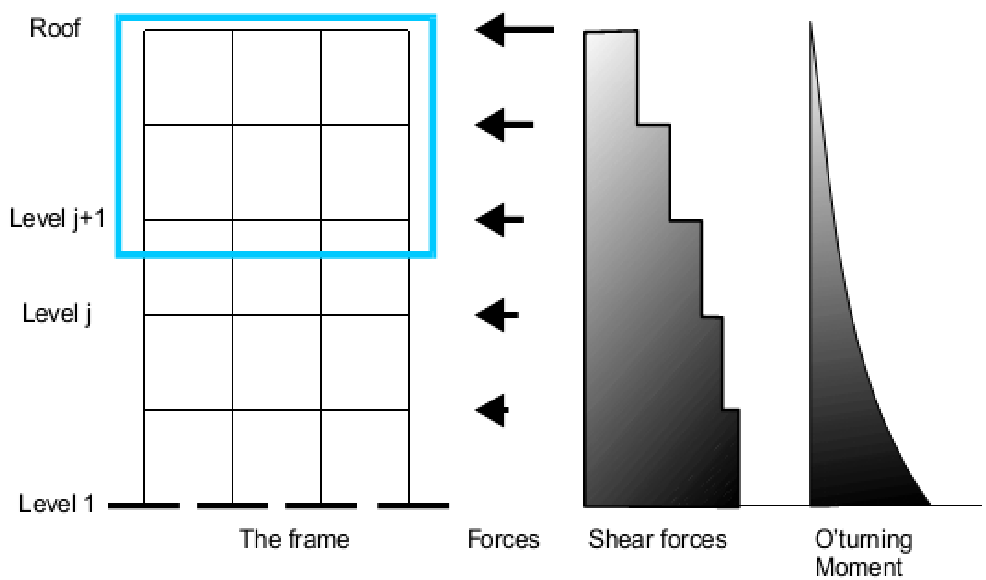


Figure 2.3 Loading Pattern and Resulting Internal Structural Actions (King, n.d)

Moehle & Mahin, (1998) observed that there are several important features of structural concepts that determine their effectiveness in resisting earthquake loading such as continuity, regularity, stiffness, proximity with adjacent buildings, mass, redundancy and previous earthquake damages. Continuity is very essential in ensuring the lateral load is transfer continuously to the foundation pad. Discontinuity usually happen as shear wall of upper level is discontinued to the lower floor thus resulting soft storey that concentrates damage.

On the other hand, sudden changes in stiffness, mass or strength of structural members also contribute to the disproportion of lateral load distribution thus causing significant torsional response to the structure. This failure could be easily observed at a particular structural member that presented certain irregularity of its structural properties. Constructing building with close proximity within each other decrease the chances of a building to swing freely during earthquake and causing adjacent buildings to pound each other and collapse. Figure 2.3 shows failure due to discontinuity and irregularity of structures.

a) b)

Figure 2.4 Structural failure due to: a) soft storey as a result of discontinuity and b) irregularity of a structural member (Moehle & Mahil, 1998)

	Damage category	Extent of damage in general	Suggested post- earthquake actions
0	No damage	No damage	No action required
I	Slighty non-structural damage	Thin cracks in plaster, falling of plaster bits in limited parts.	Building need not be vacated. Only architectural repairs needed.
II	Slight Structural Damage	Small cracks in walls, failing of plaster in large bits over large areas; damage to non-structural parts like chimneys, projecting cornices, etc. The load carrying capacity of the structure is not reduced appreciably.	Building need not be vacated. Architectural repairs required to achieve durability.
III	Moderate structural damage	Large and deep cracks in walls; widespread cracking of walls, columns, piers and tilting or failing of chimneys. The load carrying capacity of the structure is partially reduced.	Building needs to be vacated, to be reoccupied after restoration and strengthening. Structural restoration and seismic strengthening are necessary after which architec tural treatment may be carried out.
IV	Severe structural damage	Gaps occur in walls; inner and outer walls collapse; failure of ties to separate parts of buildings. Approx. 50 % of the main structural elements fail.The building takes dangerous state.	Building has to be vacated. Either the building has to be demolished or extensive restoration and strengthening work has to be carried out before reoccupation.
V	Collapse	A large part or whole of the building collapses.	Clearing the site and reconstruction.

Table 2.1 Categories of damage (Internal Association for Eartquake Engineering, Japan, 2004)

Table 2.1 is basically showing us the classification of damages based on their physical attributes and suggested action to be taken after the earthquake. Based on the table, we can see that only damages related to the architectural elements are allowed to be repaired and reused. For damages involving the structural members, it will be considered as severe and the structures are suggested to be demolished. So, we can conclude that structural failure is usually permanent and irreversible whenever the building is compromised by earthquakes.

2.4 Design Practise in Malaysia

So far, most of the design of structures in Malaysia adopted British Standard such as BS5400, BS8110, BS5950 and other standards. Unfortunately, these standards do not have details and specific requirement on seismic load. Due to that, most of engineers use standard provided by AASHTO Specifications, Uniform Building Code or Eurocode in structural designing. However, using other countries' standard causing some complication especially in adapting right ground acceleration for our country. To cope with this issue, engineer agreed to design based on importance of the structure and severity of outcome failure. Design of Penang Bridge for example, critical enough to use higher value of ground acceleration compared to design of Bakun Hydroelectric Plant (Koong & Won, 2005)

Prior to 26 December 2004 earthquake, Institution of Engineers Malaysia (IEM) in position document approved by IEM council has outlined several short term and long run recommendations on issues regarding earthquake. Among their short term recommended initiatives are urging the need of more seismic monitoring stations in Malaysia, reviewing current Engineering Design & Construction Standards and Practices as well as suggesting the design of high rise buildings to cater for long period vibration. On the other hand, for long run, IEM has suggested the development or adoption of a suitable code of practise for construction industry with related to seismic design and also recommending the introduction of earthquake engineering education curriculum in the universities. Sensitive and important structures also are recommended to be reviewed for their vulnerability when exposing tremors. (IEM, 2005)

Seismic zone mapping has been carried out by SEER (Structural Earthquake Engineering Research) and found out that for the whole Peninsular Malaysia, the ground acceleration of 0.03g to 0.05g are recommended, while in area of East Malaysia, level of acceleration recommended increasing from Sarawak towards Sabah, due to existence of active fault in Sabah. Maximum ground acceleration design in Sabah would be 0.15g (Ngu, 2005).

In term of the design of the framework system to carry lateral loads, the designer will usually adapt to three (3) most common systems; moment resisting frames, braced frames, and shear wall. Of course for skyscrapers, more sophisticated framed tube systems and other complex framing system will be adopted. Moment resisting frames are characterized as fixed or semi-rigid connection of column and girder plane frames whereby the strength and the stiffness of the concrete are proportional to the storey height and column spacing. At the same time, slab and walls also are possible to be designed as moment resisting frames. On the other hand, a braced frame contains single diagonal x-braces and k-braces connected to resist lateral loads. However, this method is popular for steel frames whereas for concrete frames, shear wall is usually constructed. Shear wall is characterized as reinforced concrete plane elements having length and thickness to provide lateral stiffness. It may be cast in place of pre-cast. (Jayachandran, 2009)

2.5 Modelling Using Excel Spreadsheet Program and STAAD.Pro

We as users always feel so comfortable with the existence of various structural analysis softwares in market. It helps in speeding up calculations thus making analysis of structure to become more efficient compared by doing hand calculations. However, by only using Microsoft Excel, engineers will be able to design their own Spreadsheet design for structural analysis. It is not only theoretical understanding that is necessary to design an Excel Spreadsheet, but also a creativity and ability to think critically and out-of-the box in order to simplify complicated engineering equations into Spreadsheet formula (Hamid, 2012).

However, when dealing with extended and thorough analysis of structure, STAAD.Pro may be the best choice among any available softwares in market. It is basically an extremely flexible modelling tool that was revolutionised by the idea of spreadsheet, and graphically inspired by AutoCAD. STAAD.Pro also practical to be used in both concrete and steel designs, hence making it a true one-stop-structural environment. On the other hand, it is also can cover all aspects of structural engineering designed to aid specific tasks and analysis (Bentley System, Inc, 2012).

STAAD.Pro which is originally developed by Research Engineers International in Yorba Linda, CA, is a type of design software that becoming of the trend nowadays. In late 2005, after Bentley System bought Research Engineer

International, they manage to commercialise STAAD.Pro, in which at the earlier stage was used as a program restricted for educational purpose of civil and structural engineers in Iowa State University. (Subramani & Shanmigam.P, 2012)

STAAD.Pro is a comprehensive and integrated design tools that provide fast and accurate results of analysis and a powerful tool to design massive structures. Undeniable, computers reduce man hours to complete a project thus ensuring fast and efficient planning as well as accurate implementation (Venkat Professional Services, 2011).

2.6 Moment Distribution Method

Developed by Prof. Hardy Cross in 1920s in response to the highly indeterminate skyscrapers being built, Moment Distribution is an iterative method of solving indeterminate structure and was presented in a paper to the American Society of Civil Engineers (ASCE) in 1932 (Caprani, 2007). In Moment Distribution, all joints are initially assumed to be fixed against rotation, then fixed end moments (FEMs) are determined based on the configuration of loading imposed to the structural members (Thomas, n.d.). Rizwan (2010) in his book titled Theory of Indeterminate Structures elaborated step by step for Moment Distribution Method. Table 2.2 shows formula for fixed-end moments for various loading configuration and end-support.

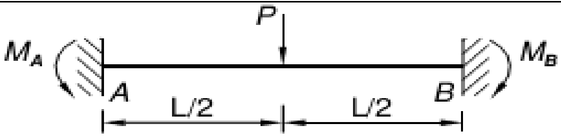
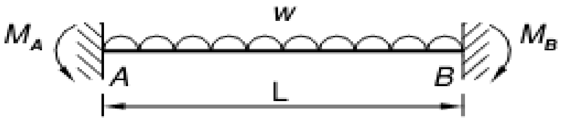
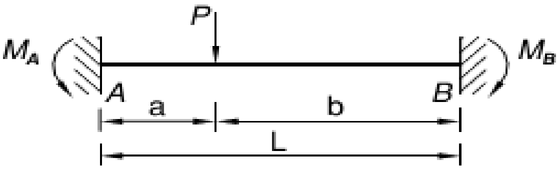
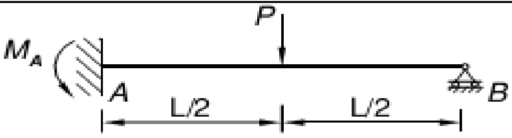
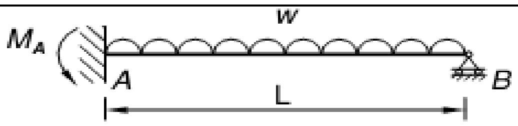
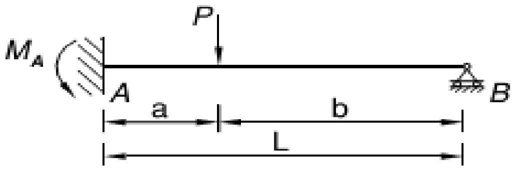
M_A	Configuration	M_B
$+\frac{PL}{8}$		$-\frac{PL}{8}$
$+\frac{wL^2}{12}$		$-\frac{wL^2}{12}$
$+\frac{Pab^2}{L^2}$		$-\frac{Pa^2b}{L^2}$
$+\frac{3PL}{16}$		-
$+\frac{wL^2}{8}$		-
$+\frac{Pab(2L-a)}{2L^2}$		-

Table 2.2 shows formula for fixed-end moments for various loading configuration and end-support (Caprani, 2007).

The first step of Moment Distribution Method is calculating fixed end moment due to applied loads. Next, relative stiffness is determined. Literally, stiffness is defined as resistance presented by member to a unit displacement or rotational for particular support conditions. After that, distribution factors for members framing at each joint are determined. Then, distribute the net fixed end moment of the joints by multiplying with respective distribution factors. In the second and subsequent cycles, moments far ends are carried over by reducing it to half. This procedure is repeated until convergence is achieved (Rizwan, 2010).

2.7 Analysing Bending Moment Diagram

It is very important to determine the bending moment profile in order to understand the moment distribution of a structure hence confirming its safety of structural element for further value of engineering where possible (Liew & Choo, 2004). Bending Moment Diagram is understood to be the total algebraic amount of forces acting on one side of the section. Sign convention is very critical in interpreting Bending Moment Diagram. As the structural member acted concavely downwards (cup-shaped), it is considered to be in sagging condition while during convexly upwards (like a hump), the member is likely to be in hogging condition. Sagging moment (positive moment) results in developing tension in bottom fibres and compression on the top while hogging moment (negative moment) produces compression on the bottom and tension on the top fibres. (Civil Engineer Educational and Industrial Resources, 2012). Figure 2.4 shows both possible bending;

Figure 2.5 Possible bending of structural members (Civil Engineer Educational and Industrial Resources, 2012)

2.8 Land Optimization and Buildings in Kuala Lumpur

The concept of high rise buildings was introduced in seventies as the first residential high rise building, 'Rifle Range' was built in Penang. Up to now, the demand for high rise living is keep on increasing and the trends are more toward quality living, where housing area are expected to have complete housing amenities like security, privacy, parking space, swimming pool, and many others (Ta, 2009). This trend actually making the developers getting more interested in mixed development along with high rise living style that are seen to be more profitable and demanded.

In Kuala Lumpur, residential land demand increased from 3,822 hectares to 5,490 hectares between 1984 and 2000 hence created many new growth areas like Wangsa Maju and Bandar Tun Razak, Damansara, Bukit Indah, Setapak and Sentul. Commercial land use also increase significantly from 504 hectares to 1,092 hectares between the same periods. The City Centre continues to be the most crowded commercial location in Kuala Lumpur which comprises 25.2 per cent of the total present commercial land use (Dewan Bandaraya Kuala Lumpur, 2012).

Due to significant increasing pressure of inhabitants, market and trade, the price of land especially in cities have levitated very high. Hence, the considerable growth in the number of high-rise buildings, for both residential and commercial is being observed. Obviously, the current trend of design is towards taller and more slender structures (Ganesan, 2003). Nowadays, the twin towers were no longer without partners. The garden area is surrounded by high rise offices, hotels and condos. Jalan Tun Razak for example, is lined with headquarters and high rise offices. Number of new towns that are surrounding the city is now developing such as Mont Kiara, Sri Kembangan, Puchong, and many others (Mohamad, 2012).

Currently, there are 663 existing buildings in Kuala Lumpur with 78 still under construction and 36 to be constructed soon. 552 of the buildings are categories as high rise buildings with height between 12*m* to 99*m* . 106 buildings are categories as skyscraper with the height between 100*m* to 452*m* whereas another 102 buildings are fall under low-rise buildings with the height between 3*m* to 11*m*. Top five tallest buildings in Kuala Lumpur are won by both Petronas Towers with the height of 452, built in 1998, followed by Menara Telekom with height of 301*m*, built in 2001. The forth tallest building in Kuala Lumpur is KLCC Lot C with height of 267*m*, finished in 2012. Lastly, Menara Maybank for the fifth tallest building in the city with the height of 244*m* built in 1988 (Emporis GMBH, 2012).

2.9 Summary

Based on thorough review from various technical papers and other types of literature, it is understood that the study on effects of lateral loading especially earthquake and wind loadings are very important to make sure that structures are prepared to face unforeseen circumstances regarding various weather conditions and natural disaster. Buildings are design as shelters to people, so their integrity, reliability and consistency are very important to ensure that they can fulfil their functions and purposes. Most part of the literature discussed on the behaviour of the frame structures as being imposed by lateral load. This understanding is essential to this study in order to check the possible failure modes and their related effects so that analysis could be done with the most accurate judgements and assumptions. On the other hand, reviewing the development and latest trend of high rise structures in Kuala Lumpur are essential to understand the parameters required and assumption to be considered while doing the modelling and analysis. It is important for the modelling to be as related as possible to the real condition and trending. In conclusions, these 7 sub-sections in literature review are very critical and influential for this analysis of existing high rise reinforced concrete structure in Malaysia with subject to earthquake and wind loadings.

CHAPTER 3

METHODOLOGY

3.1 Introduction

There are several different steps in order to complete this study. Preliminarily, this analysis requires boundaries to focus on the most critical parameters as elaborated in the first two sub-section of this chapter. The next sub-section will explain on the modelling of structure using STAAD.Pro whereas the following part will discuss on the type of analysis that will be conducted. Then, interpretation and comparison of results will be elaborated. Finally, proper recommendation and improvement method will be presented. Tools that will be used during this study are listed at the later part of this chapter.

3.2 Literature Analysis

The first step to be taken before doing thorough research regarding this topic is to critically analyse as much literature as possible corresponding to this research. This phase is important to ensure that the topic chosen is not parallel to any existing study done by other researchers. At the same time, critical analysis of technical papers, journals and reports are essential to develop deep understanding in relation to the topic. Technical knowledge and background ideas are important to ensure the right planning and suitable assumption can be applied to the topic. Various sources of literature from wide range of studies should be analysed carefully and cited for future reference. For this topic, literature analysis were done mostly related to the background information of earthquake, the behaviour of structures due to lateral load and the current trend of strata development in Kuala Lumpur.

3.3 Mapping out Research Timeline

After literature analysis was done, planning for methodology phase would need to be done. The most important preliminary element need to be developed is the project timeline along with the detailed period for each phase and activity. Mapping out the framework for the study is essential to evaluate the feasibility of the research. Problem statements and objectives of the topic were carefully analysed in order to make it balance with the allocated time for the whole research. Some preliminary objectives might be altered or reduced to fit with the timeline and framework designed based on the whole research period. The framework and timeline for each activity involved in this research are presented in the following Gantt chart and key milestone as in table 3.1 up to 3.4

No.	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14
1	**Selection of project topic**	X	X													
2	**Preliminary Research work**															
	a) Meeting and briefing with project supervisor		X													
	b) Finding relevent articles and journals			X												
	c) Preliminary research planning and structuring				X											
	d) Preparing Extended Proposal Defence					X										
3	**Submission of Extended Proposal Defence**						X									
4	**Proposal Defence**									X	X					
5	**Project works continues**															
	a) Developing spreadsheet program for structural analysis											X				
	b) Validating spreadsheet program with Staad Pro 2004												X			
	c) Preparing essential data of frame structure to be analysed													X		
6	**Submission of Interim Draft Report**														X	
7	**Submission of Interim Report**															X

Table 3.1 Gantt chart for FYP 1

No.	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14	15
1	**Project work continues**																
	a) Analysis of wind load effects on structure	X	X														
	b) Analysis of earthquake load effects on structure			X	X												
	c) Analysis of combined loads effect on structure					X	X	X									
2	**Submission of Progress Report**									X							
3	**Project work continues**																
	a) Interpret all data and results									X	X	X					
	b) Proposing modifications and solutions											X	X	X			
4	**Pre-EDX**												X				
5	**Submission of Draft Report**													X			
6	**Submission of Dissertation (soft bound)**														X		
7	**Submission of Technical Paper**														X		
8	**Oral Presentation**															X	
9	**Submission of Project Dissertation (hard bound)**																X

Table 3.2 Gantt chart for FYP 2

Week	Activities	Person Involved	Documentation Progress
	Selection of project fields and topics	FYP Coordinator	
End of Week 2	Assigned to specific supervisor	FYP Supervisor	
	Confirmation of FYP Topic by supervisor	Student	
End of Week 3	Finding Journals and do Literature Review	Student	Literature review
End of Week 4	Structuring the framework of study	Student	Methodology
End of Week 5	Preparing draft of Extended Proposal to be checked and	Student	Draft of Extended
	commended by FYP supervisor	FYP Supervisor	Proposal Defence
End of Week 6	Submission of Extended Proposal Defence	FYP Supervisor	Extended Proposal
			Defence
End of Week 9	Project Defence	Student	
		FYP Supervisor	
		Internal Examiner	
End of Week 10	Developing spreadsheet program for structural analysis	Student	
End of Week 11	Validating spreadsheet program with Staad Pro 2004	Student	
End of Week 12	Preparing essential data of frame structure to be analysed	Student	
End of Week 13	Submission of Interim Draft Report	Student	Interim Draft
		FYP Supervisor	Report
End of Week 13	Submission of Interim Report	Student	
		FYP Supervisor	Interim Report
		FYP Supervisor	

Table 3.3 Key milestone for FYP 1

Week	Activities	Person Involved	Documentation Progress
End of Week 2	Analysis of wind load effects on structure	Student	Data collection
End of Week 4	Analysis of earthquake load effects on structure	Student	Data collection
End of Week 7	Analysis of combined loads effect on structure	Student	Data collection
End of Week 8	Submission of Progress Report	FYP Supervisor	Progress Report
End of Week 10	Interpret all data and results	Student	Data and Result
		FYP Supervisor	Interpretation
		Student	
End of Week 11	Pre-EDX	FYP Supervisor	
		FYP Committee	
End of Week 12	Proposing Modifications and Solutions	Student	Discussion
	Submission of Draft Report		Draft Report
End of Week 13	Submission of Dissertation (soft bound)	FYP Supervisor	Dissertation
	Submission of technical paper	FYP Supervisor	Technical Paper
End of Week 14	Oral Presentation	Student	
		FYP Supervisor	
		FYP Committee	
End of Week 15	Submission of Project Dissertation (hard bound)	FYP Supervisor	Project
		FYP Committee	Dissertation

Table 3.4 Key milestone for FYP 2

3.4 Deciding Parameters for Analysis

After each activity and expected due date has been mapped, the first step need to be worked out just before the analysis started is listing out all the possible variables, parameters and assumptions related to the study. Of course this activity will be required to be done during the whole analysis period as assumptions could be added, removed or altered depending on the appropriateness, but preliminary elements of parameters and assumptions are essential in providing the right track for the whole study.

3.4.1 Height and Framing System

One of the important parameters to be decided upon starting the analysis is regarding the physical parameters to be considered for structural modelling. Firstly, buildings in Kuala Lumpur is categorised to different groups of height. Kuala Lumpur is chose to be the case study location as most of the high-rise structures are available here. The population in this 243.65 km^2 city is 1,800,674 which mean there are about 7,390 people in each kilometre square of area (Emporis GMBH, 2012). Crowded by both high rise structures and human population, Kuala Lumpur would be extremely sensitive in facing any possible natural disaster, so, it is clearly very significant to study structures in this area compared to any other part of Malaysia. Figure 3.1 shows the distribution of buildings in Kuala Lumpur with respect to height.

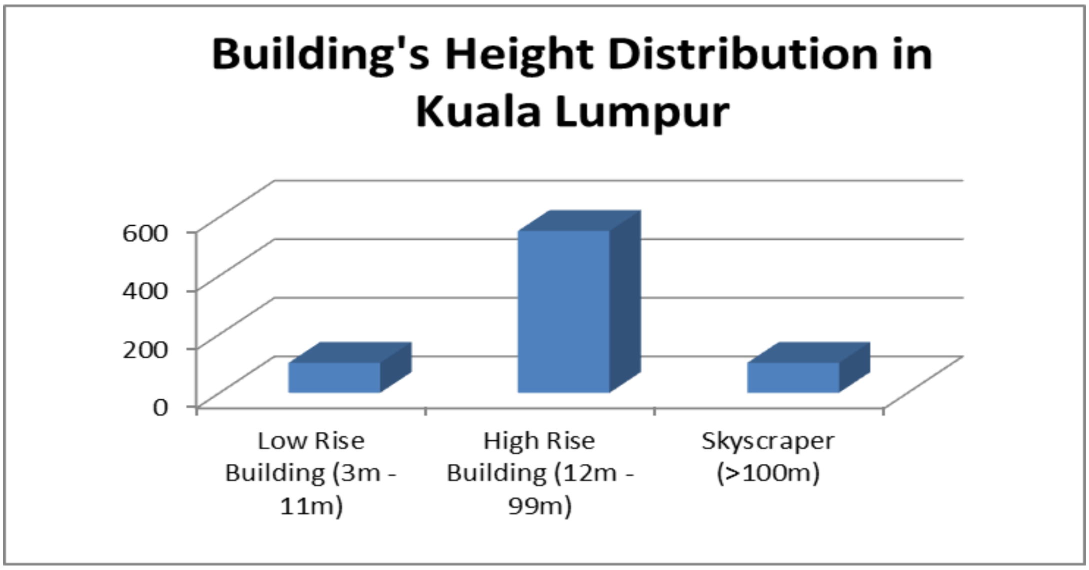


Figure 3.1 Building's Height Distribution in Kuala Lumpur

Based on the distribution in Figure 3.1, most of the buildings in Kuala Lumpur fall under high-rise building with the height between 12m to 99m. So, analysis will be done critically under this group. For low rise buildings, the effects of lateral load may be considered minimum while for skyscrapers, it is assumed that usually lateral loads have already been taken into account during designing the structural members and framing systems (Wikipedia, the free encyclopedia, 2012).

Another important parameter need to be considered is regarding the framing system to be simulated during the analysis. Type of framing system should be kept constant so that the behaviour of structures towards the lateral loads can be fully associated with the height. Different types of framing systems usually carry different level of stiffness and flexibility. Figure 3.2 shows that each type of framing system is having their own possible maximum number of storeys.

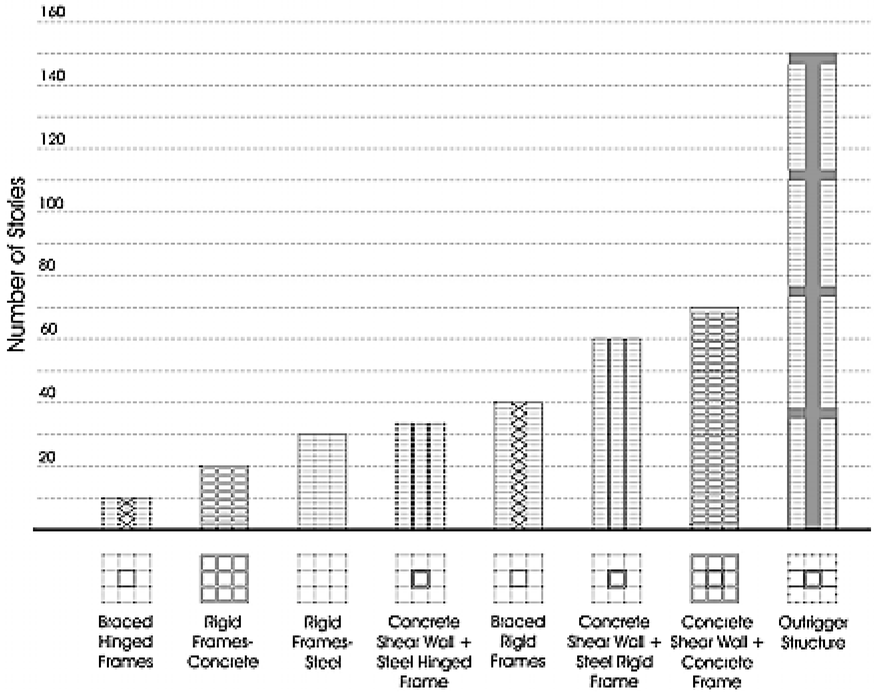


Figure 3.2 Relationship between Maximum no of stories and Type of Framing System (Ali & Moon, 2007)

As this study is concerning existing high rise concrete structures, typical rigid frames concrete is assumed, where it was widely and conservatively used before concrete shear wall becoming popular in high rise buildings. However, the above relationship is taken from the perspective of academician. Many conservative consultants, when it comes to practical height of rigid frames concrete, said that this framing system is practical to be built up to 25 storeys as long as the higher grade of concrete is used. Considering the buildings are mixed function, the floor to floor height would be 3.5*m,* so the maximum height for this rigid frame concrete would be 88*m* (CTBUH, 2011). This height is fall on the range of 12*m* to 99*m* as discussed at the earlier part of this sub-section.

3.4.2 Base Area and Member Sizing

As the range of height of structures to be analysed have been determined, the floor area of the structures also need to be assumed. To avoid more complicated parameters and variables, the base areas of the structures are first assumed to be square. Three different base areas are assumed (24*m* x 24*m*, 30*m* x 30*m*, 36*m* x 36*m*) to see the relationship between aspect ratio and structural displacement where column-to-column distance is assumed to be 6*m*. As the length of each span is same, all beam sizing also will be assumed the same with 0.15*m* thickness and 0.45*m* depth. All slabs thickness is assumed to be 0.15*m*. However, column sizes are varies due to height and summarised as in table 3.5. All columns are assumed square and sizing are based on interpolation and extrapolation of column sizing from real structures.

structure	function	floors	height (m)	column sizing (mm)	Beam sizing (m)	Slab thicness (m)
1	mixed	3	10.5	400	0.15 x 0.45	0.15
2	mixed	5	17.5	500	0.15 x 0.45	0.15
3	mixed	7	24.5	650	0.15 x 0.45	0.15
4	mixed	10	35.5	1 to 4 floors - 850 4 to 10 floors - 700	0.15 x 0.45	0.15
5	mixed	12	42	1 to 3 floors - 850 4 to 8 floors - 700 9 to 12 floors - 500	0.15 x 0.45	0.15
6	mixed	15	37.5	1 to 5 floors - 950 6 to 10 floors - 800 11 to 15 floors - 600	0.15 x 0.45	0.15
7	mixed	17	59.5	1 to 4 floors - 1100 5 to 9 floors - 950 10 to 14 floors - 800 15 to 17 floors - 550	0.15 x 0.45	0.15
8	mixed	20	70	1 to 4 floors -1200 5 to 9 floors - 1000 10 to 14 floors - 850 15 to 17 floors - 600 18 to 20 floors - 500	0.15 x 0.45	0.15
9	mixed	22	77	1 to 4 floors -1300 5 to 9 floors - 1100 10 to 14 floors - 1000 15 to 17 floors - 850 18 to 22 floors - 750	0.15 x 0.45	0.15
10	mixed	25	87.5	1 to 3 floors -1450 4 to 8 floors - 1200 9 to 13 floors - 1000 13 to 16 floors - 850 17 to 21 floors - 700 22 to 25 floors - 550	0.15 x 0.45	0.15

Table 3.5 Members' sizing for modelling

3.4.3 Loadings

Basically, there are two (2) general types of loading involved in this analysis namely vertical loads and horizontal loads. Vertical loads are characterized as loadings that are governed by gravitational force. This is including the self-weight of the structure itself, dead load as well as live load. On the other hand, horizontal load or sometimes referred as lateral loads are loadings that are imposed perpendicularly to the gravitational force. This is including wind load and earthquake load that acting side-by-side instead of upward-downward. Dead load and live load for all slabs are assumed to be 2 kN/m^2 based on British Standard. Self-weight of structure and loading from brick walls are calculated based on density of concrete and bricks that are 24 kN/m^3 and 22 kN/m^3 respectively. More details regarding loading assumptions for lateral loads (wind and earthquake) will be presented thoroughly in the 'Modelling' section.

3.5 Modelling Using STAAD.Pro 2004

Based on all parameters and variables discussed on previous sub-section, frame structures are modelled using STAAD.Pro 2004. Based on table 3.5, ten (10) frame structures are modelled with different floors. Then, the same set of models is repeated by changing the base area of the structure by putting one additional span. So, there will be three (3) set of models with different base area (24*m* x 24*m*, 30*m* x 30*m*, & 36*m* x 36*m*), whereby each model consist of different storey of buildings (3,5,10,12,15,17,20,22 & 25). So, there will be 27 different types of structural frame models that will be imposed by two (2) different lateral loads, wind and earthquake using static analysis.

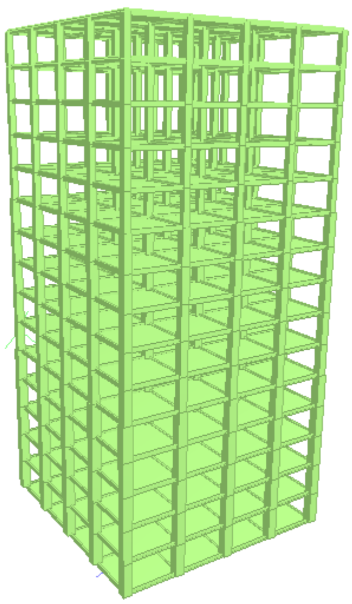


Figure 3.3 3D-rendered View of 15-Storey Model

3.6 Analysis

A dynamic analysis is required whenever inertial forces from structural accelerations are both significant and vary rapidly in time. In this study, earthquake and wind loadings are analysed dynamically. Dynamic load is basically more significant compared to static load of the same magnitude as the structure unable to respond quickly to the loading by deflecting. The increase of effect on dynamic load is given by dynamic amplification factor (DAF). DAF is calculated by dividing the maximum deflection with static deflection of the structure (Wikipedia, The Free Encyclopedia, 2012).

On the other hand, as the inertial forces are proportional to structure's mass and acceleration as stated in Newton's Law, the loads that vary slowly enough the inertial forces will be small and the response will be considered quasis-static. Sometimes, large inertial loads can be analysed using static analysis as the loads vary slowly with time. e.g. Gravity loading. However, in this study, only static analysis will be used.

3.6.1 Static analysis of wind loading

Static approach of wind loading analysis is done with reference to UBC 1997. Based on the definition of exposure condition, Kuala Lumpur is assumed to fall under exposure B which characterised as having terrain with buildings, forest or surface irregularities, covering at least 20 per cent of the ground level area extending 1 mile (1.61 km) or more from the site. Design for basic wind speed for Kuala Lumpur area is 35.1 m/s, peak 3-second gust at 10m above grade for a 50-year return period (Shafii & Othman, 2004). Based on this basic wind speed, this analysis will be using value lower than 35 m/s and higher than 35 m/s. The modelled frame structure will be imposed by different set of wind speeds from 20 m/s up to 50 m/s with constant increment of 5 m/s where 35 m/s is fall into the middle of the range (20 m/s, 25 m/s, 30 m/s, 35 m/s , 40 m/s, 45 m/s, 50 m/s).

However, before the loadings are imposed to the structure, the basic wind speed must be firstly being converted into equivalent lateral loads that acted at each floor of the structure. Design of wind pressure, p can be determined using the following formula;

$$p = C_e C_q q_s I_w$$

Where;

C_e = Combined height, exposure and gust factor coefficient as given in Table 16-G in UBC 1997

C_q = Pressure coefficient for the structure or portion of structure under consideration as given in Table 16-H in UBC 1997

I_w = Importance factor as set forth in Table 16-K in UBC 1997

p = Design wind pressure

The following tables from 3.6 up to 3.8 show table 16-G, 16-H and 16-K respectively extracted from UBC 1997. Noted, that the height is in feet.

TABLE 16-G—COMBINED HEIGHT, EXPOSURE AND GUST FACTOR COEFFICIENT (C_e)[1]

HEIGHT ABOVE AVERAGE LEVEL OF ADJOINING GROUND (feet) × 304.8 for mm	EXPOSURE D	EXPOSURE C	EXPOSURE B
0-15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13
120	1.93	1.67	1.20
160	2.02	1.79	1.31
200	2.10	1.87	1.42
300	2.23	2.05	1.63
400	2.34	2.19	1.80

[1]Values for intermediate heights above 15 feet (4572 mm) may be interpolated.

Table 3.6 Combined Heights, Exposure and Gust Factor Coefficient

TABLE 16-H—PRESSURE COEFFICIENTS (C_q)

STRUCTURE OR PART THEREOF	DESCRIPTION	C_q FACTOR
1. Primary frames and systems	**Method 1** (Normal force method)	
	Walls:	
	Windward wall	0.8 inward
	Leeward wall	0.5 outward
	Roofs[1]:	
	Wind perpendicular to ridge	
	Leeward roof or flat roof	0.7 outward
	Windward roof	
	less than 2:12 (16.7%)	0.7 outward
	Slope 2:12 (16.7%) to less than 9:12 (75%)	0.9 outward or 0.3 inward
	Slope 9:12 (75%) to 12:12 (100%)	0.4 inward
	Slope > 12:12 (100%)	0.7 inward
	Wind parallel to ridge and flat roofs	0.7 outward
	Method 2 (Projected area method)	
	On vertical projected area	
	Structures 40 feet (12 192 mm) or less in height	1.3 horizontal any direction
	Structures over 40 feet (12 192 mm) in height	1.4 horizontal any direction
	On horizontal projected area[1]	0.7 upward
2. Elements and components not in areas of discontinuity[2]	Wall elements	
	All structures	1.2 inward
	Enclosed and unenclosed structures	1.2 outward
	Partially enclosed structures	1.6 outward
	Parapets walls	1.3 inward or outward
	Roof elements[3]	
	Enclosed and unenclosed structures	
	Slope < 7:12 (58.3%)	1.3 outward
	Slope 7:12 (58.3%) to 12:12 (100%)	1.3 outward or inward
	Partially enclosed structures	
	Slope < 2:12 (16.7%)	1.7 outward
	Slope 2:12 (16.7%) to 7:12 (58.3%)	1.6 outward or 0.8 inward
	Slope > 7:12 (58.3%) to 12:12 (100%)	1.7 outward or inward
3. Elements and components in areas of discontinuities[2,4,5]	Wall corners[6]	1.5 outward or 1.2 inward
	Roof eaves, rakes or ridges without overhangs[6]	
	Slope < 2:12 (16.7%)	2.3 upward
	Slope 2:12 (16.7%) to 7:12 (58.3%)	2.6 outward
	Slope > 7:12 (58.3%) to 12:12 (100%)	1.6 outward
	For slopes less than 2:12 (16.7%)	
	Overhangs at roof eaves, rakes or ridges, and canopies	0.5 added to values above
4. Chimneys, tanks and solid towers	Square or rectangular	1.4 any direction
	Hexagonal or octagonal	1.1 any direction
	Round or elliptical	0.8 any direction
5. Open-frame towers[7,8]	Square and rectangular	
	Diagonal	4.0
	Normal	3.6
	Triangular	3.2
6. Tower accessories (such as ladders, conduit, lights and elevators)	Cylindrical members	
	2 inches (51 mm) or less in diameter	1.0
	Over 2 inches (51 mm) in diameter	0.8
	Flat or angular members	1.3
7. Signs, flagpoles, lightpoles, minor structures[8]		1.4 any direction

[1]For one story or the top story of multistory partially enclosed structures, an additional value of 0.5 shall be added to the outward C_q. The most critical combination shall be used for design. For definition of partially enclosed structures, see Section 1616.

[2]C_q values listed are for 10-square-foot (0.93 m^2) tributary areas. For tributary areas of 100 square feet (9.29 m^2), the value of 0.3 may be subtracted from C_q, except for areas at discontinuities with slopes less than 7 units vertical in 12 units horizontal (58.3% slope) where the value of 0.8 may be subtracted from C_q. Interpolation may be used for tributary areas between 10 and 100 square feet (0.93 m^2 and 9.29 m^2). For tributary areas greater than 1,000 square feet (92.9 m^2), use primary frame values.

[3]For slopes greater than 12 units vertical in 12 units horizontal (100% slope), use wall element values.

[4]Local pressures shall apply over a distance from the discontinuity of 10 feet (3048 mm) or 0.1 times the least width of the structure, whichever is smaller.

[5]Discontinuities at wall corners or roof ridges are defined as discontinuous breaks in the surface where the included interior angle measures 170 degrees or less.

[6]Load is to be applied on either side of discontinuity but not simultaneously on both sides.

[7]Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.

[8]Factors for cylindrical elements are two thirds of those for flat or angular elements.

Table 3.7 Pressure Coefficients

As mention before, Kuala Lumpur is assumed to be in exposure B, so, height is converted into feet, and coefficients are picked based on interpolation from value given in table 3.6 (Table 16-G in UBC 1997). From table 3.7, method 1 (normal force method) is assumed based on primary frames and systems whereby the coefficient of 0.8 is taken for inward and 0.5 for leeward. For importance factor in table 3.8 (Table 16-K from UBC 1997), the building is considered as standard occupancy structures which bring the factor to 1.

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5

Table 3.8 Occupancy Category

The factor of q_s in the equation is calculated using the following formula;

q_s (kN/m^2) = $0.000612v^2$

Where;

v= Speed of wind where in this case will be 20 m/s, 25 m/s, 30 m/s, 35 m/s, 40 m/s, 45 m/s and 50 m/s

Then, based on the above formula and coefficients, the lateral load acted on each floor of buildings are tabulated into table using Excel Spreadsheet and presented as the following tables;

Level	Height, h (m)	Height,h (ft)	q_s (kN/m^2)	C_e		C_q		Pressure, p (Mpa) kN/m^2		
				WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	0.24496	0.62	1.603	0.8	-0.5	0.122	0.196	0.318
2	7	22.96	0.24496	0.7	1.603	0.8	-0.5	0.137	0.196	0.334
3	10.5	34.44	0.24496	0.8	1.603	0.8	-0.5	0.157	0.196	0.353
4	14	45.92	0.24496	0.873	1.603	0.8	-0.5	0.171	0.196	0.367
5	17.5	57.4	0.24496	0.94	1.603	0.8	-0.5	0.184	0.196	0.381
6	21	68.88	0.24496	0.991	1.603	0.8	-0.5	0.194	0.196	0.391
7	24.5	80.36	0.24496	1.05	1.603	0.8	-0.5	0.206	0.196	0.402
8	28	91.84	0.24496	1.094	1.603	0.8	-0.5	0.214	0.196	0.411
9	31.5	103.32	0.24496	1.144	1.603	0.8	-0.5	0.224	0.196	0.421
10	35	114.8	0.24496	1.183	1.603	0.8	-0.5	0.232	0.196	0.428
11	38.5	126.28	0.24496	1.219	1.603	0.8	-0.5	0.239	0.196	0.435
12	42	137.76	0.24496	1.25	1.603	0.8	-0.5	0.245	0.196	0.441
13	45.5	149.24	0.24496	1.296	1.603	0.8	-0.5	0.254	0.196	0.450
14	49	160.72	0.24496	1.33	1.603	0.8	-0.5	0.261	0.196	0.457
15	52.5	172.2	0.24496	1.346	1.603	0.8	-0.5	0.264	0.196	0.460
16	56	183.68	0.24496	1.376	1.603	0.8	-0.5	0.270	0.196	0.466
17	59.5	195.16	0.24496	1.409	1.603	0.8	-0.5	0.276	0.196	0.472
18	63	206.64	0.24496	1.435	1.603	0.8	-0.5	0.281	0.196	0.478
19	66.5	218.12	0.24496	1.46	1.603	0.8	-0.5	0.286	0.196	0.482
20	70	229.6	0.24496	1.483	1.603	0.8	-0.5	0.291	0.196	0.487
21	73.5	241.08	0.24496	1.509	1.603	0.8	-0.5	0.296	0.196	0.492
22	77	252.56	0.24496	1.531	1.603	0.8	-0.5	0.300	0.196	0.496
23	80.5	264.04	0.24496	1.557	1.603	0.8	-0.5	0.305	0.196	0.501
24	84	275.52	0.24496	1.58	1.603	0.8	-0.5	0.310	0.196	0.506
25	87.5	287	0.24496	1.603	1.603	0.8	-0.5	0.314	0.196	0.510

Table 3.9 Equivalent Lateral Load for Wind Speed of 20 m/s

Level	Height, h (m)	Height,h (ft)	q_s (kN/m^2)	C_e		C_q		Pressure, p (Mpa) kN/m^2		
				WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	0.38275	0.62	1.603	0.8	-0.5	0.190	0.307	0.497
2	7	22.96	0.38275	0.7	1.603	0.8	-0.5	0.214	0.307	0.521
3	10.5	34.44	0.38275	0.8	1.603	0.8	-0.5	0.245	0.307	0.552
4	14	45.92	0.38275	0.873	1.603	0.8	-0.5	0.267	0.307	0.574
5	17.5	57.4	0.38275	0.94	1.603	0.8	-0.5	0.288	0.307	0.595
6	21	68.88	0.38275	0.991	1.603	0.8	-0.5	0.303	0.307	0.610
7	24.5	80.36	0.38275	1.05	1.603	0.8	-0.5	0.322	0.307	0.628
8	28	91.84	0.38275	1.094	1.603	0.8	-0.5	0.335	0.307	0.642
9	31.5	103.32	0.38275	1.144	1.603	0.8	-0.5	0.350	0.307	0.657
10	35	114.8	0.38275	1.183	1.603	0.8	-0.5	0.362	0.307	0.669
11	38.5	126.28	0.38275	1.219	1.603	0.8	-0.5	0.373	0.307	0.680
12	42	137.76	0.38275	1.25	1.603	0.8	-0.5	0.383	0.307	0.690
13	45.5	149.24	0.38275	1.296	1.603	0.8	-0.5	0.397	0.307	0.704
14	49	160.72	0.38275	1.33	1.603	0.8	-0.5	0.407	0.307	0.714
15	52.5	172.2	0.38275	1.346	1.603	0.8	-0.5	0.412	0.307	0.719
16	56	183.68	0.38275	1.376	1.603	0.8	-0.5	0.421	0.307	0.728
17	59.5	195.16	0.38275	1.409	1.603	0.8	-0.5	0.431	0.307	0.738
18	63	206.64	0.38275	1.435	1.603	0.8	-0.5	0.439	0.307	0.746
19	66.5	218.12	0.38275	1.46	1.603	0.8	-0.5	0.447	0.307	0.754
20	70	229.6	0.38275	1.483	1.603	0.8	-0.5	0.454	0.307	0.761
21	73.5	241.08	0.38275	1.509	1.603	0.8	-0.5	0.462	0.307	0.769
22	77	252.56	0.38275	1.531	1.603	0.8	-0.5	0.469	0.307	0.776
23	80.5	264.04	0.38275	1.557	1.603	0.8	-0.5	0.477	0.307	0.784
24	84	275.52	0.38275	1.58	1.603	0.8	-0.5	0.484	0.307	0.791
25	87.5	287	0.38275	1.603	1.603	0.8	-0.5	0.491	0.307	0.798

Table 3.10 Equivalent Lateral Load for Wind Speed of 25 m/s

Level	Height, h (m)	Height,h (ft)	q_s (kN/m^2)	C_e		C_q		Pressure, p (Mpa) kN/m^2		
				WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	0.55116	0.62	1.603	0.8	-0.5	0.273	0.442	0.715
2	7	22.96	0.55116	0.7	1.603	0.8	-0.5	0.309	0.442	0.750
3	10.5	34.44	0.55116	0.8	1.603	0.8	-0.5	0.353	0.442	0.794
4	14	45.92	0.55116	0.873	1.603	0.8	-0.5	0.385	0.442	0.827
5	17.5	57.4	0.55116	0.94	1.603	0.8	-0.5	0.414	0.442	0.856
6	21	68.88	0.55116	0.991	1.603	0.8	-0.5	0.437	0.442	0.879
7	24.5	80.36	0.55116	1.05	1.603	0.8	-0.5	0.463	0.442	0.905
8	28	91.84	0.55116	1.094	1.603	0.8	-0.5	0.482	0.442	0.924
9	31.5	103.32	0.55116	1.144	1.603	0.8	-0.5	0.504	0.442	0.946
10	35	114.8	0.55116	1.183	1.603	0.8	-0.5	0.522	0.442	0.963
11	38.5	126.28	0.55116	1.219	1.603	0.8	-0.5	0.537	0.442	0.979
12	42	137.76	0.55116	1.25	1.603	0.8	-0.5	0.551	0.442	0.993
13	45.5	149.24	0.55116	1.296	1.603	0.8	-0.5	0.571	0.442	1.013
14	49	160.72	0.55116	1.33	1.603	0.8	-0.5	0.586	0.442	1.028
15	52.5	172.2	0.55116	1.346	1.603	0.8	-0.5	0.593	0.442	1.035
16	56	183.68	0.55116	1.376	1.603	0.8	-0.5	0.607	0.442	1.048
17	59.5	195.16	0.55116	1.409	1.603	0.8	-0.5	0.621	0.442	1.063
18	63	206.64	0.55116	1.435	1.603	0.8	-0.5	0.633	0.442	1.074
19	66.5	218.12	0.55116	1.46	1.603	0.8	-0.5	0.644	0.442	1.086
20	70	229.6	0.55116	1.483	1.603	0.8	-0.5	0.654	0.442	1.096
21	73.5	241.08	0.55116	1.509	1.603	0.8	-0.5	0.665	0.442	1.107
22	77	252.56	0.55116	1.531	1.603	0.8	-0.5	0.675	0.442	1.117
23	80.5	264.04	0.55116	1.557	1.603	0.8	-0.5	0.687	0.442	1.128
24	84	275.52	0.55116	1.58	1.603	0.8	-0.5	0.697	0.442	1.138
25	87.5	287	0.55116	1.603	1.603	0.8	-0.5	0.707	0.442	1.149

Table 3.11 Equivalent Lateral Load for Wind Speed of 30 m/s

Level	Height, h (m)	Height,h (ft)	q_s (kN/m^2)	C_e		C_q		Pressure, p (Mpa) kN/m^2		
				WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	0.75019	0.62	1.603	0.8	-0.5	0.372	0.601	0.973
2	7	22.96	0.75019	0.7	1.603	0.8	-0.5	0.420	0.601	1.021
3	10.5	34.44	0.75019	0.8	1.603	0.8	-0.5	0.480	0.601	1.081
4	14	45.92	0.75019	0.873	1.603	0.8	-0.5	0.524	0.601	1.125
5	17.5	57.4	0.75019	0.94	1.603	0.8	-0.5	0.564	0.601	1.165
6	21	68.88	0.75019	0.991	1.603	0.8	-0.5	0.595	0.601	1.196
7	24.5	80.36	0.75019	1.05	1.603	0.8	-0.5	0.630	0.601	1.231
8	28	91.84	0.75019	1.094	1.603	0.8	-0.5	0.657	0.601	1.258
9	31.5	103.32	0.75019	1.144	1.603	0.8	-0.5	0.687	0.601	1.288
10	35	114.8	0.75019	1.183	1.603	0.8	-0.5	0.710	0.601	1.311
11	38.5	126.28	0.75019	1.219	1.603	0.8	-0.5	0.732	0.601	1.333
12	42	137.76	0.75019	1.25	1.603	0.8	-0.5	0.750	0.601	1.351
13	45.5	149.24	0.75019	1.296	1.603	0.8	-0.5	0.778	0.601	1.379
14	49	160.72	0.75019	1.33	1.603	0.8	-0.5	0.798	0.601	1.399
15	52.5	172.2	0.75019	1.346	1.603	0.8	-0.5	0.808	0.601	1.409
16	56	183.68	0.75019	1.376	1.603	0.8	-0.5	0.826	0.601	1.427
17	59.5	195.16	0.75019	1.409	1.603	0.8	-0.5	0.846	0.601	1.447
18	63	206.64	0.75019	1.435	1.603	0.8	-0.5	0.861	0.601	1.462
19	66.5	218.12	0.75019	1.46	1.603	0.8	-0.5	0.876	0.601	1.477
20	70	229.6	0.75019	1.483	1.603	0.8	-0.5	0.890	0.601	1.491
21	73.5	241.08	0.75019	1.509	1.603	0.8	-0.5	0.906	0.601	1.507
22	77	252.56	0.75019	1.531	1.603	0.8	-0.5	0.919	0.601	1.520
23	80.5	264.04	0.75019	1.557	1.603	0.8	-0.5	0.934	0.601	1.536
24	84	275.52	0.75019	1.58	1.603	0.8	-0.5	0.948	0.601	1.550
25	87.5	287	0.75019	1.603	1.603	0.8	-0.5	0.962	0.601	1.563

Table 3.12 Equivalent Lateral Load for Wind Speed of 35 m/s

Level	Height, h	Height,h	q_s	C_e		C_q		Pressure, p (Mpa) kN/m^2		
	(m)	(ft)	(kN/m^2)	WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	0.97984	0.62	1.603	0.8	-0.5	0.486	0.785	1.271
2	7	22.96	0.97984	0.7	1.603	0.8	-0.5	0.549	0.785	1.334
3	10.5	34.44	0.97984	0.8	1.603	0.8	-0.5	0.627	0.785	1.412
4	14	45.92	0.97984	0.873	1.603	0.8	-0.5	0.684	0.785	1.470
5	17.5	57.4	0.97984	0.94	1.603	0.8	-0.5	0.737	0.785	1.522
6	21	68.88	0.97984	0.991	1.603	0.8	-0.5	0.777	0.785	1.562
7	24.5	80.36	0.97984	1.05	1.603	0.8	-0.5	0.823	0.785	1.608
8	28	91.84	0.97984	1.094	1.603	0.8	-0.5	0.858	0.785	1.643
9	31.5	103.32	0.97984	1.144	1.603	0.8	-0.5	0.897	0.785	1.682
10	35	114.8	0.97984	1.183	1.603	0.8	-0.5	0.927	0.785	1.713
11	38.5	126.28	0.97984	1.219	1.603	0.8	-0.5	0.956	0.785	1.741
12	42	137.76	0.97984	1.25	1.603	0.8	-0.5	0.980	0.785	1.765
13	45.5	149.24	0.97984	1.296	1.603	0.8	-0.5	1.016	0.785	1.801
14	49	160.72	0.97984	1.33	1.603	0.8	-0.5	1.043	0.785	1.828
15	52.5	172.2	0.97984	1.346	1.603	0.8	-0.5	1.055	0.785	1.840
16	56	183.68	0.97984	1.376	1.603	0.8	-0.5	1.079	0.785	1.864
17	59.5	195.16	0.97984	1.409	1.603	0.8	-0.5	1.104	0.785	1.890
18	63	206.64	0.97984	1.435	1.603	0.8	-0.5	1.125	0.785	1.910
19	66.5	218.12	0.97984	1.46	1.603	0.8	-0.5	1.144	0.785	1.930
20	70	229.6	0.97984	1.483	1.603	0.8	-0.5	1.162	0.785	1.948
21	73.5	241.08	0.97984	1.509	1.603	0.8	-0.5	1.183	0.785	1.968
22	77	252.56	0.97984	1.531	1.603	0.8	-0.5	1.200	0.785	1.985
23	80.5	264.04	0.97984	1.557	1.603	0.8	-0.5	1.220	0.785	2.006
24	84	275.52	0.97984	1.58	1.603	0.8	-0.5	1.239	0.785	2.024
25	87.5	287	0.97984	1.603	1.603	0.8	-0.5	1.257	0.785	2.042

Table 3.13 Equivalent Lateral Load for Wind Speed of 40 m/s

Level	Height, h	Height,h	q_s	C_e		C_q		Pressure, p (Mpa) kN/m^2		
	(m)	(ft)	(kN/m^2)	WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	1.24011	0.62	1.603	0.8	-0.5	0.615	0.994	1.609
2	7	22.96	1.24011	0.7	1.603	0.8	-0.5	0.694	0.994	1.688
3	10.5	34.44	1.24011	0.8	1.603	0.8	-0.5	0.794	0.994	1.788
4	14	45.92	1.24011	0.873	1.603	0.8	-0.5	0.866	0.994	1.860
5	17.5	57.4	1.24011	0.94	1.603	0.8	-0.5	0.933	0.994	1.927
6	21	68.88	1.24011	0.991	1.603	0.8	-0.5	0.983	0.994	1.977
7	24.5	80.36	1.24011	1.05	1.603	0.8	-0.5	1.042	0.994	2.036
8	28	91.84	1.24011	1.094	1.603	0.8	-0.5	1.085	0.994	2.079
9	31.5	103.32	1.24011	1.144	1.603	0.8	-0.5	1.135	0.994	2.129
10	35	114.8	1.24011	1.183	1.603	0.8	-0.5	1.174	0.994	2.168
11	38.5	126.28	1.24011	1.219	1.603	0.8	-0.5	1.209	0.994	2.203
12	42	137.76	1.24011	1.25	1.603	0.8	-0.5	1.240	0.994	2.234
13	45.5	149.24	1.24011	1.296	1.603	0.8	-0.5	1.286	0.994	2.280
14	49	160.72	1.24011	1.33	1.603	0.8	-0.5	1.319	0.994	2.313
15	52.5	172.2	1.24011	1.346	1.603	0.8	-0.5	1.335	0.994	2.329
16	56	183.68	1.24011	1.376	1.603	0.8	-0.5	1.365	0.994	2.359
17	59.5	195.16	1.24011	1.409	1.603	0.8	-0.5	1.398	0.994	2.392
18	63	206.64	1.24011	1.435	1.603	0.8	-0.5	1.424	0.994	2.418
19	66.5	218.12	1.24011	1.46	1.603	0.8	-0.5	1.448	0.994	2.442
20	70	229.6	1.24011	1.483	1.603	0.8	-0.5	1.471	0.994	2.465
21	73.5	241.08	1.24011	1.509	1.603	0.8	-0.5	1.497	0.994	2.491
22	77	252.56	1.24011	1.531	1.603	0.8	-0.5	1.519	0.994	2.513
23	80.5	264.04	1.24011	1.557	1.603	0.8	-0.5	1.545	0.994	2.539
24	84	275.52	1.24011	1.58	1.603	0.8	-0.5	1.567	0.994	2.561
25	87.5	287	1.24011	1.603	1.603	0.8	-0.5	1.590	0.994	2.584

Table 3.14 Equivalent Lateral Load for Wind Speed of 45 m/s

Level	Height, h (m)	Height,h (ft)	q_s (kN/m^2)	C_e		C_q		Pressure, p (Mpa) kN/m^2		
				WW	LW	WW	LW	WW	LW	Total
1	3.5	11.48	1.531	0.62	1.603	0.8	-0.5	0.759	1.227	1.986
2	7	22.96	1.531	0.7	1.603	0.8	-0.5	0.857	1.227	2.084
3	10.5	34.44	1.531	0.8	1.603	0.8	-0.5	0.980	1.227	2.207
4	14	45.92	1.531	0.873	1.603	0.8	-0.5	1.069	1.227	2.296
5	17.5	57.4	1.531	0.94	1.603	0.8	-0.5	1.151	1.227	2.378
6	21	68.88	1.531	0.991	1.603	0.8	-0.5	1.214	1.227	2.441
7	24.5	80.36	1.531	1.05	1.603	0.8	-0.5	1.286	1.227	2.513
8	28	91.84	1.531	1.094	1.603	0.8	-0.5	1.340	1.227	2.567
9	31.5	103.32	1.531	1.144	1.603	0.8	-0.5	1.401	1.227	2.628
10	35	114.8	1.531	1.183	1.603	0.8	-0.5	1.449	1.227	2.676
11	38.5	126.28	1.531	1.219	1.603	0.8	-0.5	1.493	1.227	2.720
12	42	137.76	1.531	1.25	1.603	0.8	-0.5	1.531	1.227	2.758
13	45.5	149.24	1.531	1.296	1.603	0.8	-0.5	1.587	1.227	2.814
14	49	160.72	1.531	1.33	1.603	0.8	-0.5	1.629	1.227	2.856
15	52.5	172.2	1.531	1.346	1.603	0.8	-0.5	1.649	1.227	2.876
16	56	183.68	1.531	1.376	1.603	0.8	-0.5	1.685	1.227	2.912
17	59.5	195.16	1.531	1.409	1.603	0.8	-0.5	1.726	1.227	2.953
18	63	206.64	1.531	1.435	1.603	0.8	-0.5	1.758	1.227	2.985
19	66.5	218.12	1.531	1.46	1.603	0.8	-0.5	1.788	1.227	3.015
20	70	229.6	1.531	1.483	1.603	0.8	-0.5	1.816	1.227	3.043
21	73.5	241.08	1.531	1.509	1.603	0.8	-0.5	1.848	1.227	3.075
22	77	252.56	1.531	1.531	1.603	0.8	-0.5	1.875	1.227	3.102
23	80.5	264.04	1.531	1.557	1.603	0.8	-0.5	1.907	1.227	3.134
24	84	275.52	1.531	1.58	1.603	0.8	-0.5	1.935	1.227	3.162
25	87.5	287	1.531	1.603	1.603	0.8	-0.5	1.963	1.227	3.190

Table 3.15 Equivalent Lateral Load for Wind Speed of 50 m/s

Based on lateral load obtained from tables, analyses are done using STAAD.Pro for different height and base area of frame structures as modelled before. Figure 3.4 shows how lateral load for 50 m/s of wind speed are imposed for each floor for 15 storey building using the software.

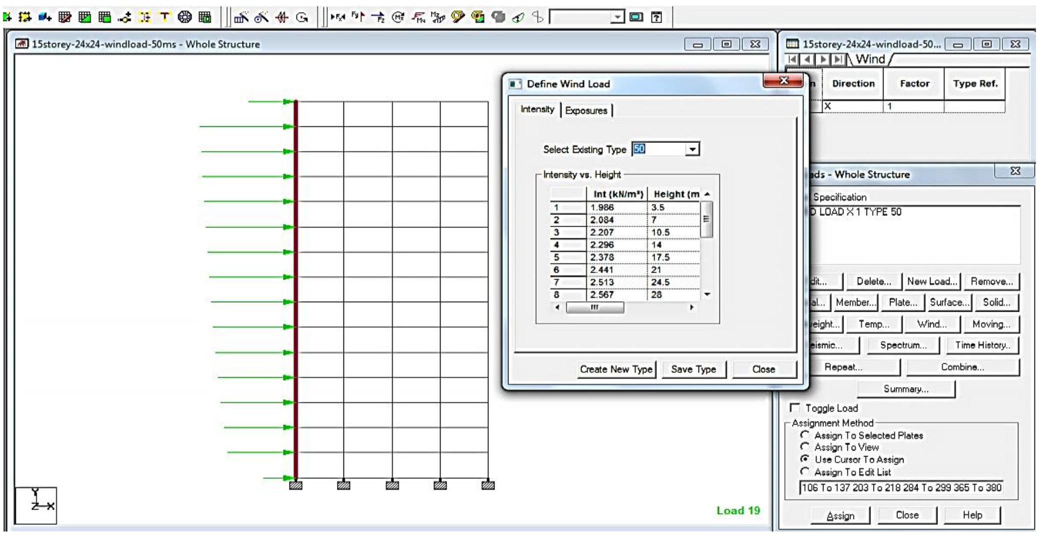


Figure 3.4 Applying Wind Load to 15-storey Frame Structure

3.6.2 Static Analysis of Earthquake Loading

Like wind loading, earthquake loading also is analysed based on UBC 1997. However, unlike wind loading that is basically acting toward the surface area between floor of the structure, earthquake loading is acting toward the mass between the floors. While wind loading is directly related with the height of the structure, earthquake loading is basically governed by the total mass of the structure. So, the first step in analysing the earthquake loading is by determining the total mass of the structure as well as mass for each floor. This can be done by specifying all sizing of structural members thus converting it into mass of structure. Noted, that live load is excluded considering the earthquake may occur whenever occupant is not in the building. So, it is more critical to analyse it just by considering the self-weight plus dead load of the structure. Based on table 3.16, seismic factor is determined whereby Malaysia is fall under zone 2A and seismic factor of 0.15 is considered.

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Note: The zone shall be determined from the seismic zone map.

Table 3.16 Seismic Factor

Occupancy Category	Seismic Importance Factor, I
1-Essential facilities	1.25
2-Hazardous facilities	1.25
3-Special occupancy structures	1.00
4-Standard occupancy structures	1.00
5-Miscellaneous structures	1.00

Table 3.17 Importance Factor of Seismic Load

After seismic zone is determined, like wind loading, importance factor need to be decided based on the function and occupancy of the structure. Here, importance factor of 1 is considered. Then, R factor need to be determined based on framing system as shown in table 3.18. For this analysis, ordinary moment resisting frame system is assumed which carries the R factor of 3.5.

Basic Structural System	Lateral- force resisting system description	R	$\Omega_{\circ}$	Height limit Zones 3 &4. (meters)
Bearing Wall	Concrete shear walls	4.5	2.8	48
Building Frame	Concrete shear walls	5.5	2.8	73
Moment-Resisting Frame	SMRF IMRF OMRF	8.5 5.5 3.5	2.8 2.8 2.8	N.L ---- ----
Dual	Shear wall + SMRF Shear wall + IMRF	8.5 6.5	2.8 2.8	N.L 48
Cantilevered Column Building	Cantilevered column elements	2.2	2.0	10
Shear-wall Frame Interaction		5.5	2.8	48

Table 3.18 R-Factor

Apart from that, the performance of structure toward earthquake loading is also dependent on the type of soil where the structure is located. Stronger soil will bring lower coefficient means lower amplification of ground acceleration compared to softer soil. The C_v and C_a factors are summarised as in table 3.19 and 3.20. In this analysis, the soil type is assumed to be type SA where C_v and C_a both carry the coefficient of 0.12 and 0.15 respectively.

Soil Profile Type	Seismic Zone Factor, Z				
	Z =0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_a
S_B	0.08	0.15	0.20	0.30	0.40 N_a
S_C	0.13	0.25	0.33	0.45	0.56 N_a
S_D	0.18	0.32	0.40	0.54	0.64 N_a
S_E	0.26	0.50	0.64	0.84	0.96 N_a
S_F	See Footnote				

Table 3.19 Seismic Coefficient, C_v

Soil Profile Type	Seismic Zone Factor, Z				
	Z =0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_a
S_B	0.08	0.15	0.20	0.30	0.40 N_a
S_C	0.09	0.18	0.24	0.33	0.40 N_a
S_D	0.12	0.22	0.28	0.36	0.44 N_a
S_E	0.19	0.30	0.34	0.36	0.36 N_a
S_F	See Footnote				

Table 3.20 Seismic Coefficient, C_a

Then, period of the structure will be calculated based on the following formula;

$$T = C_t h_n^{3/4}$$

Where;

T = Period of the structure

C_t = For RC frame is 0.0731

h_n = Total height of building in meters

Then, using period calculated, total weight of structure and coefficients obtained from table 3.16 to 3.20, base shear of the structure can be calculated using formula;

$$V = \frac{Cv\ I\ W}{R\ T}$$

Where;

It cannot exceed

$$V = \frac{2.5\ Ca\ I\ W}{R}$$

And should not be less than

$$V = 0.11\ Ca\ I\ W$$

Table 3.21 shows the calculation of weight for each floor based on member sizing as assumed from the earlier part of analysis. Multiplying the weight per floor with number of floors will obtain the total mass of the structure. On the other hand, Table 3.22 summarised the base shear of particular storeys. Noted, all base shear obtained are within the minimum and maximum value which are proven by 'TRUE' remarks resulting from logical function developed in Excel Spreadsheet.

Average Column Size (m)	:	0.8	m	Width of Building (m)	:	24	
No of Column	:	25		Weight of column per floor	:	1344	kN
Length of Beam each floor	:	240	m	Weight of Beam per floor	:	388.8	kN
Area of Slab each floor	:	576	m^2	Weight of Slab per floor	:	3225.6	kN
Length of Brickwall each floor	:	96	m	Weight of Brickwall per floor	:	1848	kN
Length of partitions per floor	:	144	m	Weight of Partition per floor	:	288	kN
				Total Weight per floor	:	7094.4	kN
				weight of roof	:	3614.4	kN

Table 3.21 Weight of Structure

no of storey	height(m)	period	W	Base Shear		Vmax	Vmin	remarks
3	10.5	0.4264	24897.6	2001.987	kN	2134.08	328.6483	TRUE
5	17.5	0.6255	39086.4	2142.609	kN	3350.263	515.9405	TRUE
7	24.5	0.8050	53275.2	2269.062	kN	4566.446	703.2326	TRUE
10	35	1.0519	74558.4	2430.196	kN	6390.72	984.1709	TRUE
12	42	1.2060	88747.2	2522.978	kN	7606.903	1171.463	TRUE
15	52.5	1.4257	110030.4	2645.995	kN	9431.177	1452.401	TRUE
17	59.5	1.5660	124219.2	2719.549	kN	10647.36	1639.693	TRUE
20	70	1.7691	145502.4	2819.957	kN	12471.63	1920.632	TRUE
22	77	1.9001	159691.2	2881.435	kN	13687.82	2107.924	TRUE
25	87.5	2.0913	180974.4	2966.927	kN	15512.09	2388.862	TRUE

Table 3.22 Summary of Base Shear

Based on base shear obtain, lateral load that acting on each level of the structure is calculated. Basically, base shear is converted into lateral load by depending on ratio between the mass of a particular floor with the total mass of the structure. That is why, higher level of floor will carry higher weightage as they are consist of higher mass compared to lower level of the floor. Here in static analysis, the increment of weightage from lower part to higher part of structure is assumed constant whereby in dynamic analysis later on, we will see that the increment of lateral load through the height of a structure may not supposed to be constant. Table 3.23 shows one example of equivalent lateral load for 15-storey structure.

15 storey F_t: 264.07 $V\text{-}F_t$: 2381.92

Level	hi (m)	wi (kN)	wihi (kNm)	(wxhx)/z(wihi)	F (kN)
roof	52.5	3614.40	189756.00	0.07	161.60
15	49	7094.40	347625.60	0.12	296.04
14	45.5	7094.40	322795.20	0.12	274.90
13	42	7094.40	297964.80	0.11	253.75
12	38.5	7094.40	273134.40	0.10	232.61
11	35.0	7094.40	248304.00	0.09	211.46
10	31.5	7094.40	223473.60	0.08	190.31
9	28.0	7094.40	198643.20	0.07	169.17
8	24.5	7094.40	173812.80	0.06	148.02
7	21.0	7094.40	148982.40	0.05	126.88
6	17.5	7094.40	124152.00	0.04	105.73
5	14.0	7094.40	99321.60	0.04	84.58
4	10.5	7094.40	74491.20	0.03	63.44
3	7.0	7094.40	49660.80	0.02	42.29
2	3.5	7094.40	24830.40	0.01	21.15
			2796948.00	1.00	2381.92

Table 3.23 Seismic Lateral Load for 15-Storey Structure

3.7 Interpretation of Data

It is important to understand the art of interpreting all the data into simple and direct presentation that will be able to show all possible relationships, correlations, and interactions between all variables and parameters. At the end of the analysis, the qualitative resistance of structures towards wind and earthquake loads will be discussed in relation with the variation of the heights, the foundation areas as well as the nature of the design itself. Thorough qualitative analysis will be done in order to achieve the understandings of behaviour of high rise structures towards these loads. If necessary, mathematical equations would be developed to represent each correlation thus giving opportunities for any interpolation as well as extrapolation of data.

3.8 Discussion and Recommendation of Possible Improvement

Comparing conventional design with new design will be done once the analysis finished and completed. This part of study is important in order to understand the different in term of structural integrity between old structures and new structures. This discussion also may be lead to recommendation of suitable enhancement needed to old structure to maintain their structural integrity as new one. The improvement method may be the one already being implemented in other countries but with proper adaptation and adjustment to be used in Malaysia.

3.9 Tools

- Microsoft Excel 2010
- STAAD.Pro 2004
- UBC 1997
- IS
- BS (5950/6399)

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

At this part, all results will be presented and interpreted in such ways that it could be easily understood. Divided into four (4) sub-sections, the first one will be the result of static analysis of wind loading, followed by earthquake loading of the same approach. Then, the last two parts will be on dynamic analysis of those two loadings.

4.2 Static Analysis of Wind Loading

After the analysis is run using the STAAD.Pro, the behaviour of the structure imposed by wind loading is observed. One of the critical parameters to be taken account is the maximum displacement of the structure. Typical serviceability check also often called as *deflection index* consider the values of H/100 to H/600 for maximum buildings deflection depending on building type and material used. However, people usually assume it to be H/400 and H/500 (Griffis, 2003). So, in this analysis, maximum allowable deflection for a structure is taken as H/500, where, if there is any deflection more than this value, the structure will be assumed failed.

Apart from that, critical structural members whose having maximum stress and deflection are also being taken into consideration. The relationship between structural height and total displacement are presented graphically in the graphs.

Figure 4.1 and 4.2 shows the results obtained from STAAD.Pro analysis. Green lines in figure 4.2 show the displacement of the nodes. Table 4.1 is the summary of maximum displacement undergone by the structure for different height, base area and imposed wind loads.

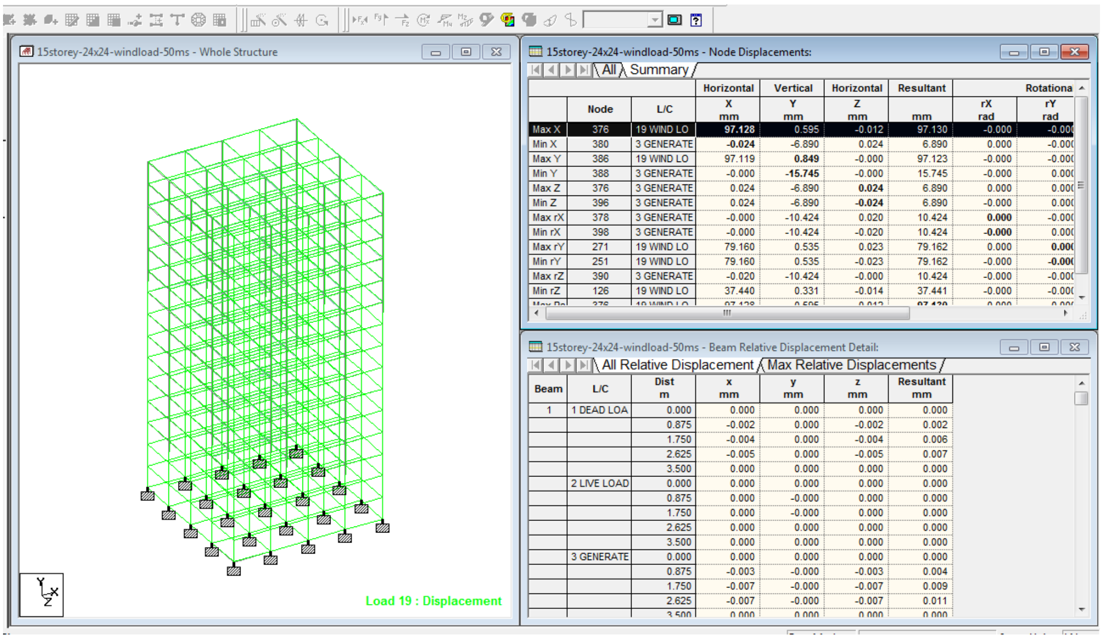


Figure 4.1 Results from STAAD.Pro Analysis

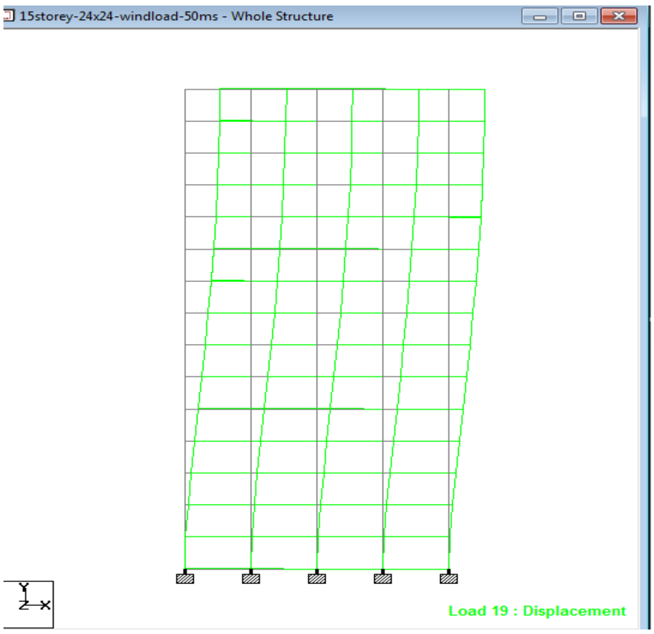


Figure 4.2 Structural Displacements Due to Wind Load

Wind Speed m/s	Storeys	Heights (m)	Max. horizontal displacement (mm) for different floor areas				Max allow.
			18m x 18m	24m x 24m	30m x 30m	36m x 36m	h/500
20	3	10.5	0.971	0.827	0.695	0.599	21
	5	17.5	2.332	1.849	1.532	1.307	35
	7	24.5	3.919	3.073	2.528	2.147	49
	10	35	-	6.073	4.985	4.644	70
	12	42	-	9.738	7.975	6.752	84
	15	52.5	-	15.539	12.702	10.74	105
	17	59.5	-	19.213	15.708	13.284	119
	20	70	-	27.782	22.69	19.174	140
	22	77	-	32.756	26.73	22.576	154
	25	87.5	-	44.048	38.287	30.308	175
25	3	10.5	1.596	1.292	1.085	0.936	21
	5	17.5	3.644	2.889	2.394	2.043	35
	7	24.5	6.121	4.8	3.984	3.353	49
	10	35	-	9.485	7.787	7.254	70
	12	42	-	15.217	12.462	10.551	84
	15	52.5	-	24.287	19.852	16.785	105
	17	59.5	-	30.028	24.55	20.761	119
	20	70	-	43.418	35.461	29.966	140
	22	77	-	51.198	41.78	35.287	154
	25	87.5	-	68.921	56.159	47.384	175
30	3	10.5	2.296	1.859	1.562	1.346	21
	5	17.5	5.245	4.159	3.445	2.941	35
	7	24.5	8.815	6.914	5.686	4.829	49
	10	35	-	13.657	11.212	10.444	70
	12	42	-	21.907	17.941	15.19	84
	15	52.5	-	34.962	28.578	24.163	105
	17	59.5	-	43.231	35.345	29.89	119
	20	70	-	62.515	51.057	43.146	140
	22	77	-	73.712	60.153	50.804	154
	25	87.5	-	99.217	80.845	68.212	175
35	3	10.5	3.126	2.531	2.126	1.833	21
	5	17.5	7.138	5.66	4.688	4.002	35
	7	24.5	11.995	9.407	7.737	6.571	49
	10	35	-	18.589	15.26	14.216	70
	12	42	-	29.818	24.419	20.675	84
	15	52.5	-	47.588	38.898	32.889	105
	17	59.5	-	58.846	48.112	40.686	119
	20	70	-	85.081	69.487	58.721	140
	22	77	-	100.323	81.868	69.145	154
	25	87.5	-	135.043	110.037	92.843	175
40	3	10.5	4.083	3.307	2.777	2.394	21
	5	17.5	9.326	7.394	6.125	5.228	35
	7	24.5	15.668	12.288	10.107	8.584	49
	10	35	-	24.282	19.934	18.569	70
	12	42	-	38.949	31.897	27.007	84
	15	52.5	-	62.161	50.809	42.961	105
	17	59.5	-	76.866	62.844	53.145	119
	20	70	-	111.144	90.773	76.708	140
	22	77	-	131.045	106.939	90.319	154
	25	87.5	-	176.392	143.731	121.272	175

Table 4.1 (a) Summary of Maximum Deflection Due to Different Wind Speed

45	3	10.5	5.169	4.186	3.516	3.031	21
	5	17.5	11.805	9.36	7.753	6.618	35
	7	24.5	19.835	15.556	12.794	10.866	49
	10	35	-	30.734	25.231	23.504	70
	12	42	-	49.297	40.371	34.181	84
	15	52.5	-	78.675	64.307	54.374	105
	17	59.5	-	97.285	79.538	67.263	119
	20	70	-	140.665	114.883	97.082	140
	22	77	-	165.859	135.349	114.314	154
	25	87.5	-	223.244	181.907	153.483	175
50	3	10.5	6.381	5.168	4.34	3.741	21
	5	17.5	14.57	11.553	9.57	8.169	35
	7	24.5	24.483	19.202	15.793	13.413	49
	10	35	-	37.939	31.146	29.013	70
	12	42	-	60.856	49.838	42.196	84
	15	52.5	-	97.128	79.391	67.128	105
	17	59.5	-	120.102	98.193	83.038	119
	20	70	-	173.656	141.828	119.852	140
	22	77	-	204.755	167.09	141.122	154
	25	87.5	-	275.597	224.566	189.476	175

Table 4.1 (b) Summary of Maximum Deflection Due to Different Wind Speed

The data is further being interpreted in graphs as shown below;

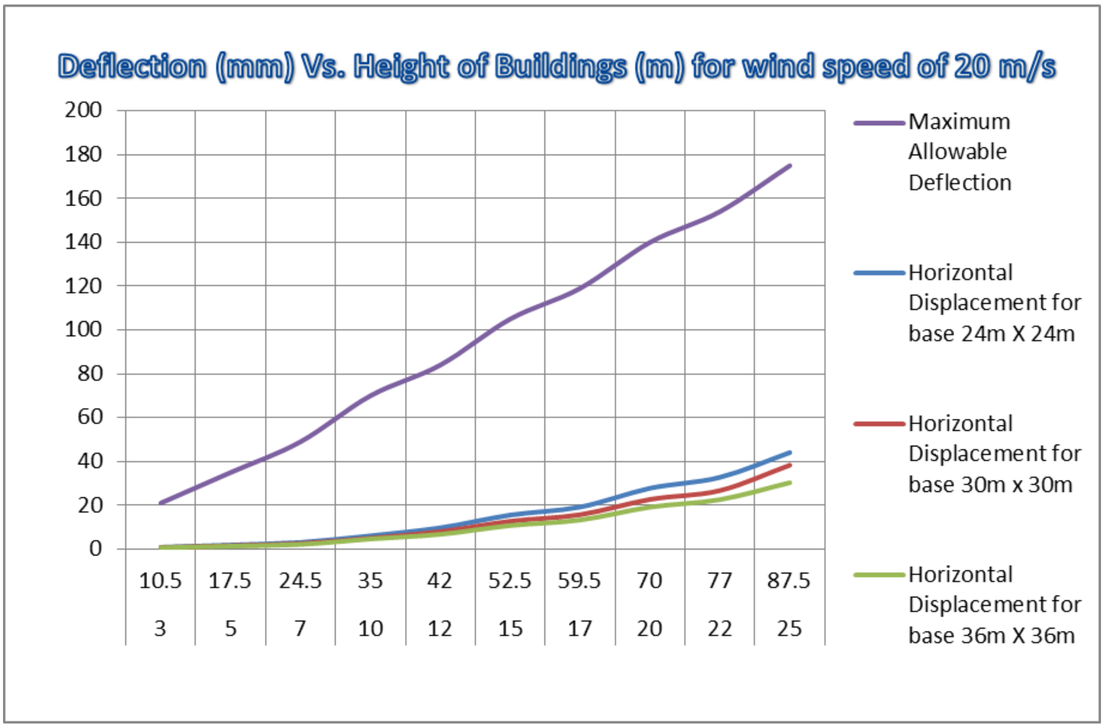


Figure 4.3 Deflection Vs. height of buildings for wind speed of 20 m/s

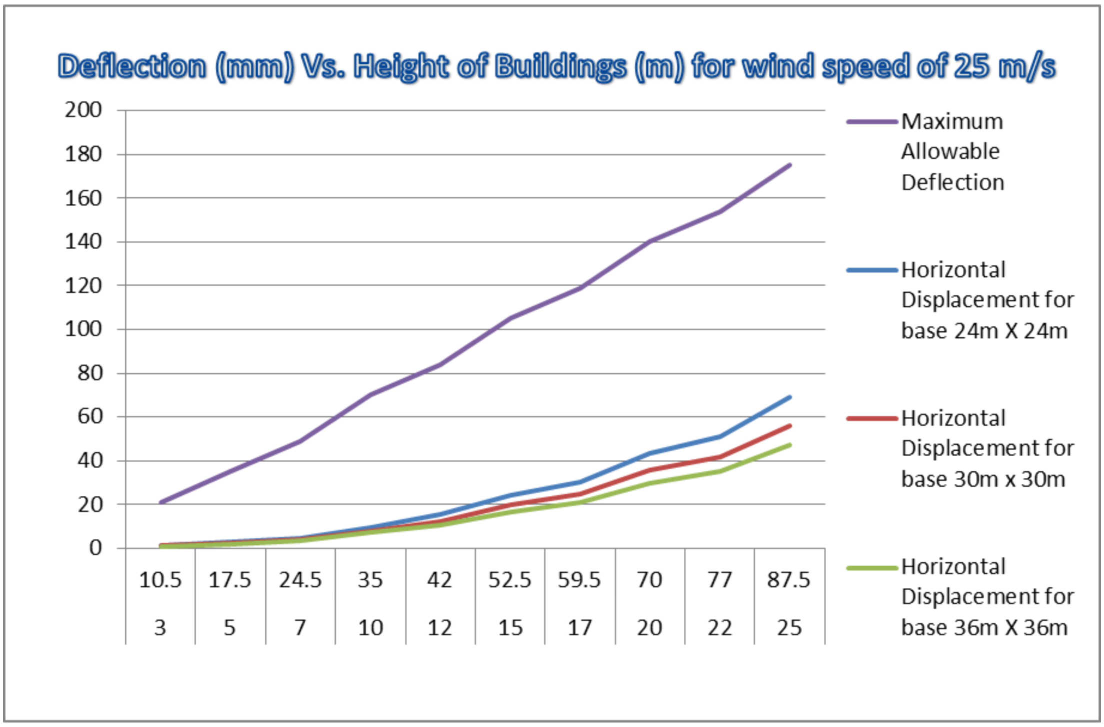


Figure 4.4 Deflection Vs. height of buildings for wind speed of 25 m/s

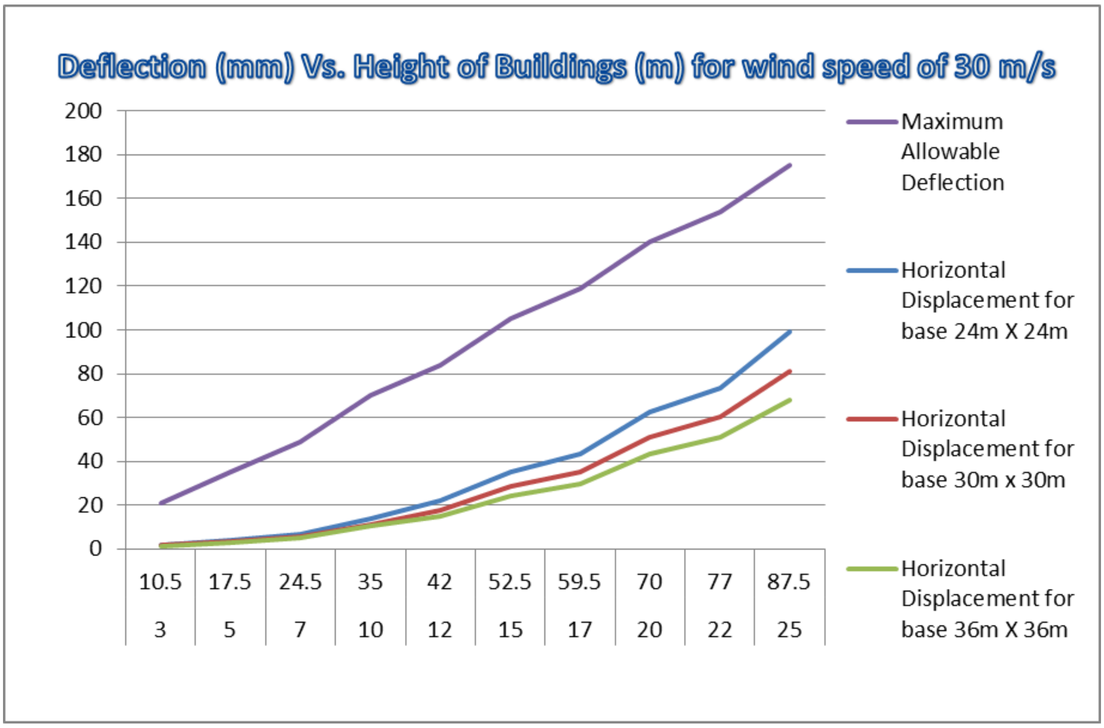


Figure 4.5 Deflection Vs. height of buildings for wind speed of 30 m/s

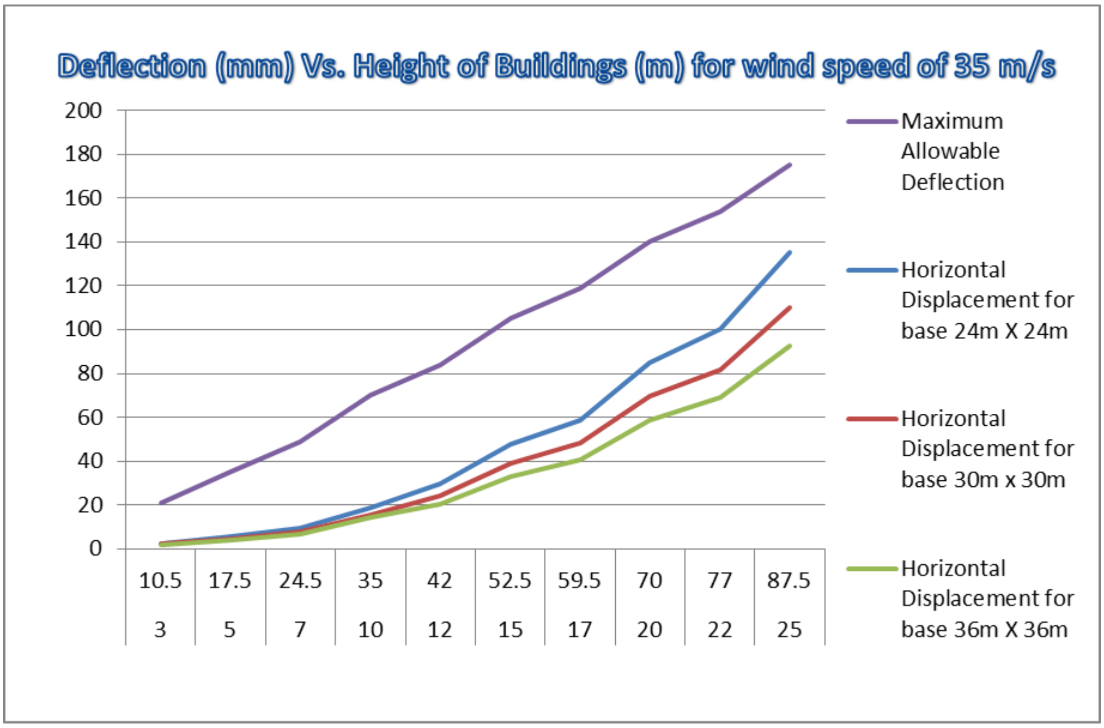


Figure 4.6 Deflection Vs. height of buildings for wind speed of 35 m/s

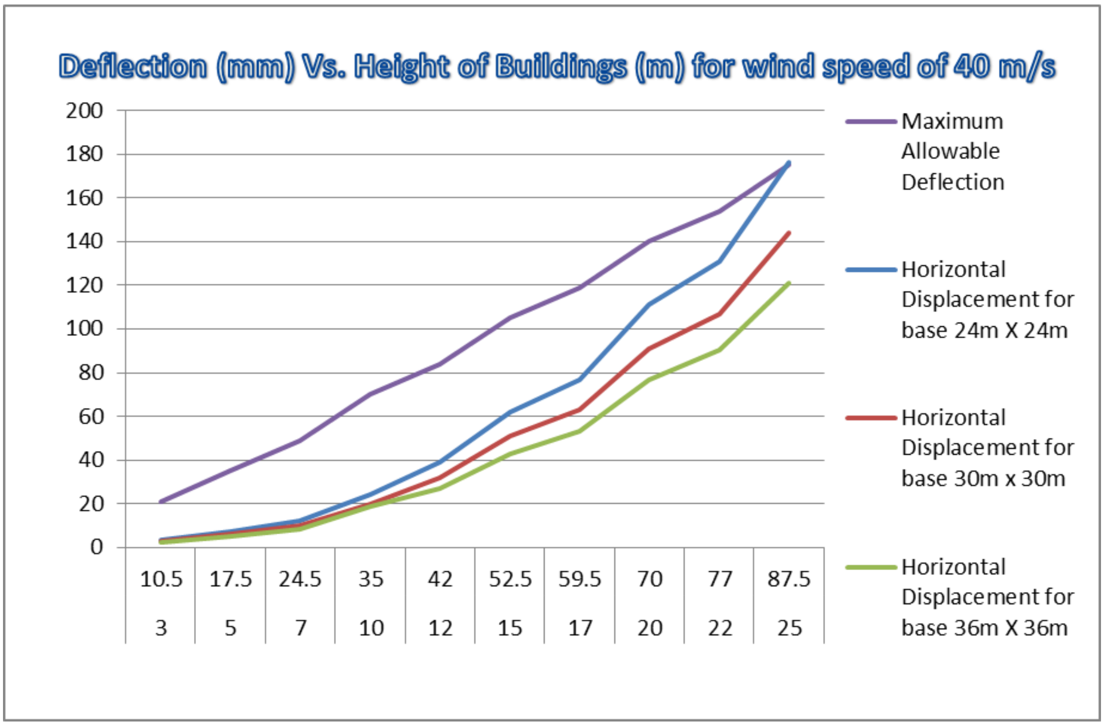


Figure 4.7 Deflection Vs. height of buildings for wind speed of 40 m/s

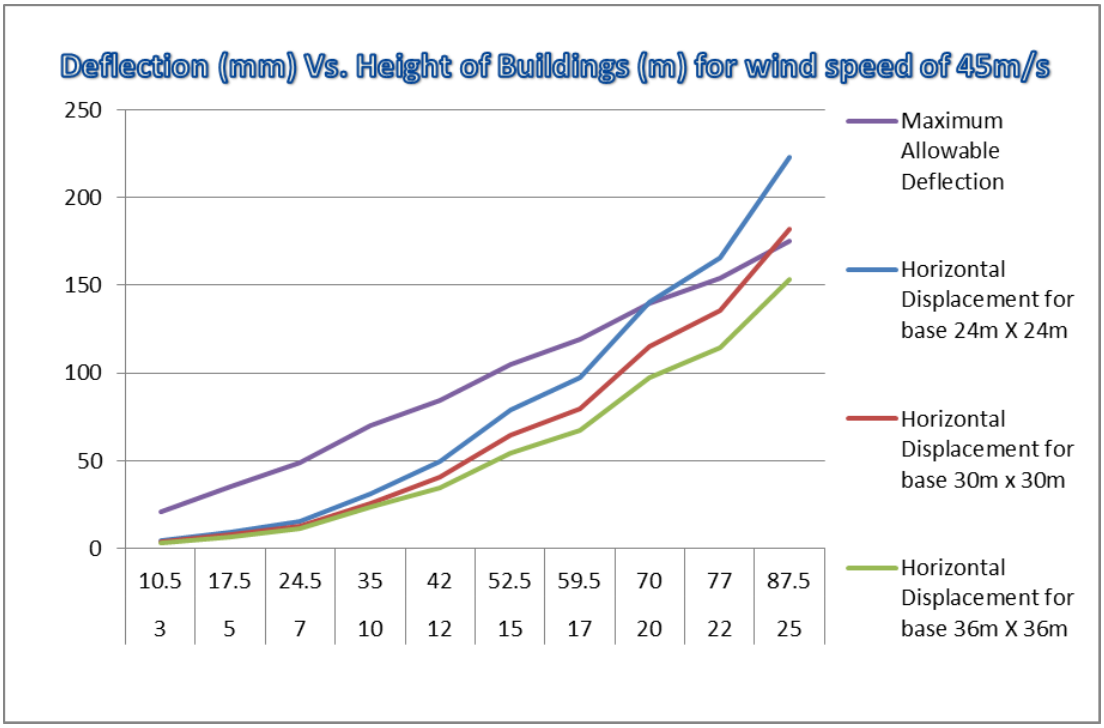


Figure 4.8 Deflection Vs. height of buildings for wind speed of 45 m/s

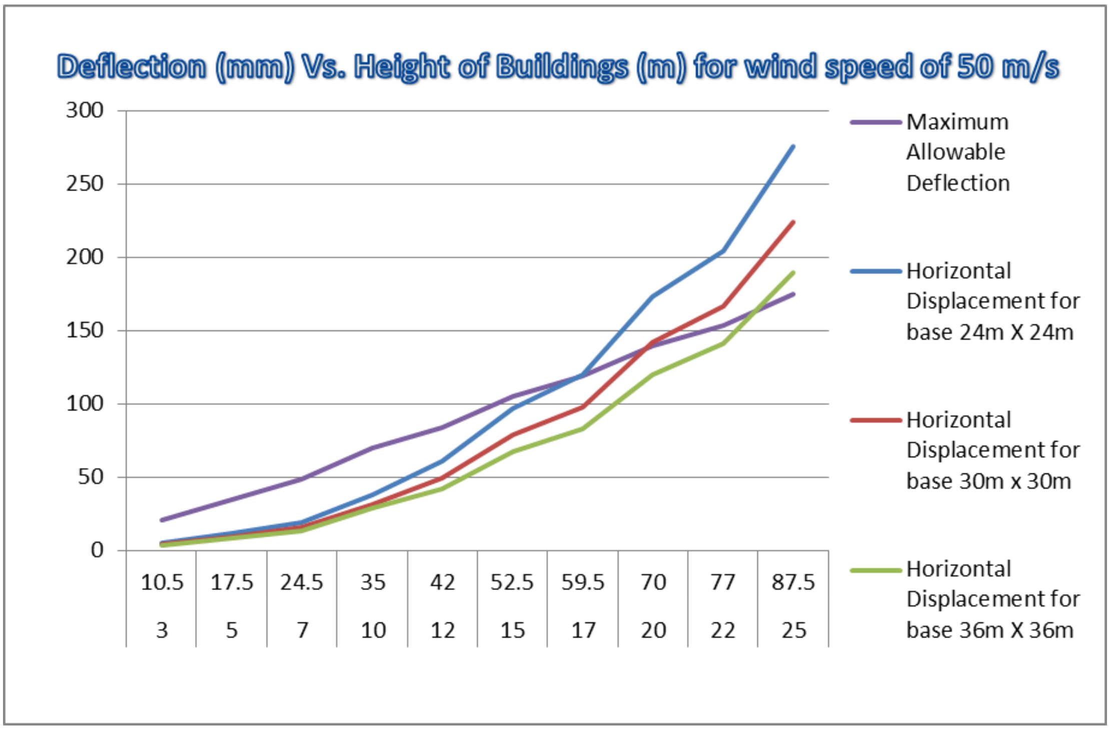


Figure 4.9 Deflection Vs. height of buildings for wind speed of 50 m/s

Based on the graphs presented, it is clearly observed that theoretically all buildings studied are safe for wind speed of 20 m/s, 25 m/s, 30 m/s, and 35 m/s even though lateral loads are excluded in the design. This is practically an answer on negligible recorded structural damage for buildings in Malaysia where maximum basic wind speed is around 32 m/s to 34 m/s. However, for wind speed of 40 m/s, maximum allowable deflection is achieved by structural with maximum practical height for moment resisting frame building with base of 24m x 24m. So, any high rise building with any dimension lower that 24m whether square or rectangular shaped will be considered no longer practical without any lateral resistance design. However, based on data taken from Official Portal of Malaysian Meteorological Department, highest maximum wind speed ever recorded was 41.7 m/s, at Kuching, Sarawak on 15 September 1992. This means for wind speed beyond 40 m/s was only recorded for the last 20 years in a place located very far away from Kuala Lumpur, where high rise buildings are congested. Later, for wind speed of 45 m/s and 50 m/s, all high rise buildings with more than 17 stories will eventually be severely affected by the lateral loads imposed.

Studies also show that low rise and medium rise buildings up to 15 stories performed very well in resisting lateral load without any additional bracing or lateral design. This means that for buildings up to 15 stories/53m, the loadings are governed by gravity instead of laterally. However, for buildings more than 15 stories, the lateral loads gives more effect compared to gravity loads based on the gradient of the displacement curve. On the other hand, the relationship and trends between the base area and the displacement of the buildings are simply interpreted based on the table and graph shown below;

No of Stories	Height (m)	Width/lenght (m)	Imposed Area,A (m2)	No. of Column	Imposed area/ No of column
	10.5	24	252	25	10.08
3	10.5	30	315	36	8.75
	10.5	36	378	49	7.71
	17.5	24	420	25	16.80
5	17.5	30	525	36	14.58
	17.5	36	630	49	12.86
	52.5	24	1260	25	50.40
15	52.5	30	1575	36	43.75
	52.5	36	1890	49	38.57
	87.5	24	2100	25	84.00
25	87.5	30	2625	36	72.92
	87.5	36	3150	49	64.29

Table 4.2 Relationship between height and resistance given by the column

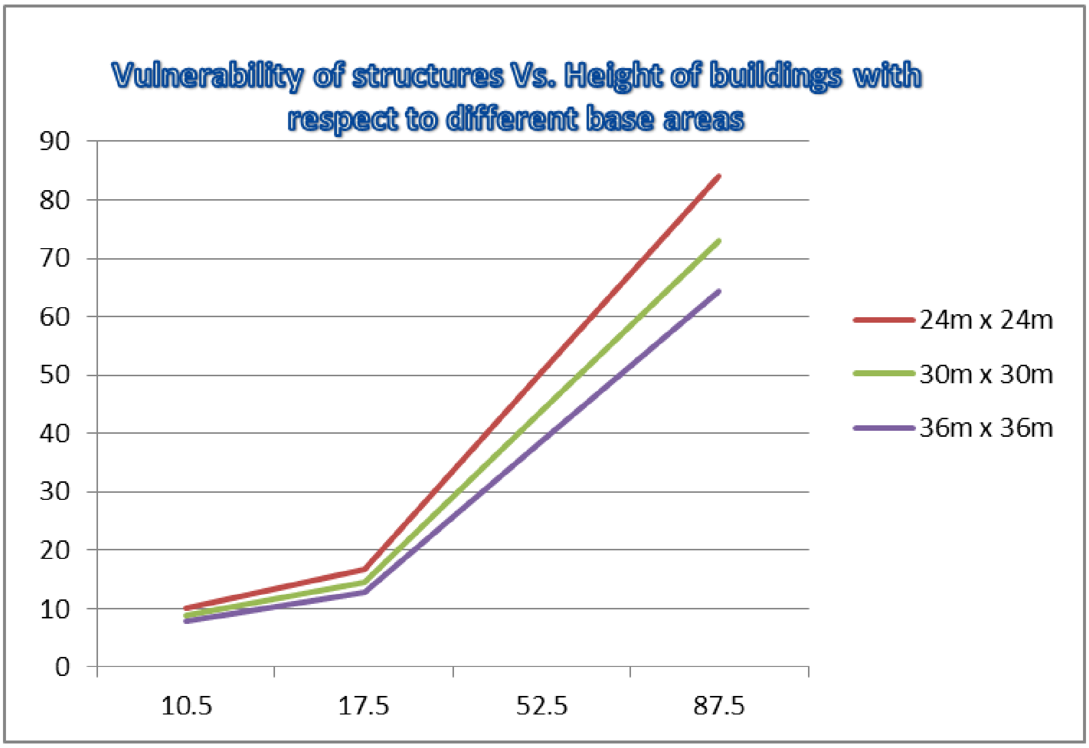


Figure 4.10 vulnerability of structure vs. height of building

Based on the table, the resistance of structure contributed by the column is represented with the number of column given in a structure. The ratio between imposed area and no of columns represent the resistance of structure towards lateral loadings. Higher ratio characterises lower vulnerability of structure towards wind loads and vice versa. The difference of resistance given for different base area also increases with height. This answers the higher differences in deflections for high rise structure compared to medium and low rise structure with respect to different base areas.

4.3 Static Analysis of Earthquake Loading

After running input from equivalent lateral load imposed due to seismic movement based on UBC 1997, the horizontal displacement of each analysed building is tabulated in table 4.3.

Storeys	Heights (m)	Max. horizontal displacement (mm) for different floor areas				Max allow.
		18m x 18m	24m x 24m	30m x 30m	36m x 36m	h/500
3	10.5	-	24.91	24.96	24.93	21
5	17.5	-	33.04	32.44	31.98	35
7	24.5	-	38.93	37.75	36.93	49
10	35	-	56.96	54.92	58.87	70
12	42	-	75.82	72.86	70.85	84
15	52.5	-	100.27	95.96	93.03	105
17	59.5	-	114.81	109.76	106.31	119
20	70	-	147.92	141.08	136.43	140
22	77	-	157.28	149.79	144.71	154
25	87.5	-	196.05	186.24	179.62	175

Table 4.3 horizontal displacement due to earthquake loading based on UBC 1997

However, the data presented in table 4.2 are based on regional condition in which already being categorised in UBC 1997. All coefficients and constants are predetermined by zoning. So, it is impossible to change the magnitude of earthquake as Malaysia is already categorised in zone 2A. As UBC 1997 is too sophisticated to be manipulated by changing the ground acceleration, IS 1893 is utilised due to its simpler and direct method in determining the equivalent lateral loads. As most of the constants and coefficients in IS 1893 is not determined by zoning criteria, a slight modification of the code making it is available for ground acceleration manipulation for the study. Using this code, only buildings with 3, 5, 15, and 25 are analyse to represent low rise buildings, middle rise as well as high rise buildings. The tabulated data is presented in table 4.4 as below;

Ground Acc. g	Storeys	Type of buildings	Heights (m)	Max. horizontal displacement (mm) for different floor areas				Max allow.
				18m x 18m	24m x 24m	30m x 30m	36m x 36m	h/500
0.05	3	Low rise	10.5	-	11.57	11.59	11.61	21
	5	Low rise	17.5	-	18.99	18.62	18.37	35
	15	Medium rise	52.5	-	96.66	92.58	89.78	105
	25	High rise	87.5	-	231.95	220.67	213.04	175
0.1	3	Low rise	10.5	-	23.14	23.18	23.22	21
	5	Low rise	17.5	-	37.97	37.23	43.07	35
	15	Medium rise	52.5	-	193.32	185.16	179.57	105
	25	High rise	87.5	-	463.91	441.34	426.06	175
0.15	3	Low rise	10.5	-	34.71	34.77	34.82	21
	5	Low rise	17.5	-	56.96	55.87	64.6	35
	15	Medium rise	52.5	-	289.98	277.73	269.37	105
	25	High rise	87.5	-	695.86	662.03	639.08	175
0.2	3	Low rise	10.5	-	46.28	46.36	46.43	21
	5	Low rise	17.5	-	75.94	74.49	86.13	35
	15	Medium rise	52.5	-	386.63	370.31	359.16	105
	25	High rise	87.5	-	927.81	882.68	852.11	175

Table 4.4 horizontal displacement due to earthquake loading based on IS 1893

Table 4.3 is further being plotted into figure 4.11 in order to observe the trends and patterns of the deflections with respect to the height of the structures.

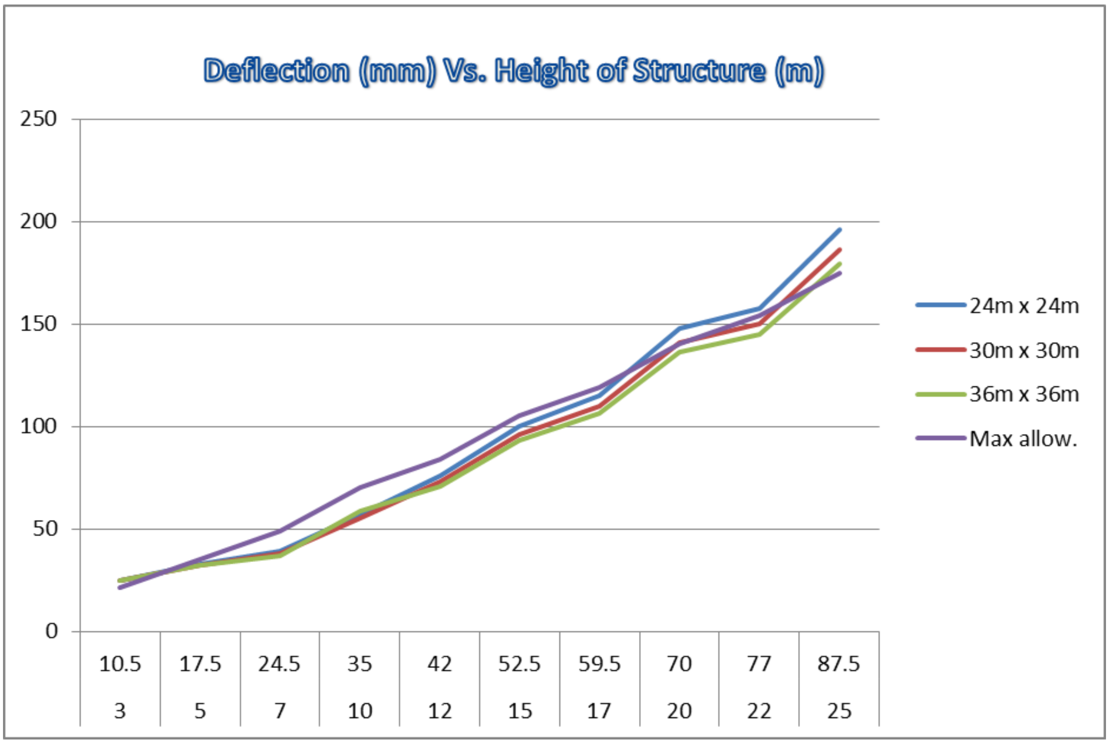


Figure 4.11 Deflection vs. height of the structures

Based on figure 4.11 plotted with respect to behaviour of structure according to UBC 1997, it is clearly observed that very low rise structures tend to have more effect toward seismic loading compared to medium rise structure. Then, high rise structure theoretically will have deflection slightly beyond their allowance. However, UBC 1997 only shows the behaviour of structures in consideration of seismic zoning location which is pre-determined in the code. In order to see more clearly on the effects of buildings toward different magnitude of ground accelerations, table 4.4, horizontal displacement due to seismic according to IS 1893 is referred.

Based on table 4.4, low rise and medium rise buildings seem to be able to survive in ground acceleration of 0.05g. SEER mapping shows that Peninsular Malaysia so far only experience ground acceleration between 0.03g to 0.05g. Kuala Lumpur might not yet experiencing 0.05g of ground acceleration since none structural damage is reported due to earthquake so far. However, the study suggests that structural improvement for existing high rise with more than 77 meter height whereby designed by only considering gravity loadings need to be implemented. On the other hand, any ground acceleration beyond 0.1g will cause severe damages to all moment resisting frame structure regardless their height.

Having looked at both UBC 1997 and IS 1893, behaviour of structures toward seismic load are independent with their respective base areas. Unlike wind loading where lateral load is represented by imposed area, seismic load is characterized by the weight of the whole structure. Lighter structures with lower base shear usually survive to seismic load. As number of column usually proportionate to the weight of the building, proportional resistance also given by the column to resist seismic load. That explains the differences of deflection with respect to base areas are much lower compared to wind loading.

4.4 Summary

In order to deduce and providing simpler arguments regarding behaviour of the structures, it is important to interpret all raw data into graphical forms as shown throughout this chapter. Reasoning and observation has been made and discussed with supervisor to ensure all explanations are valid based on theoretical understanding and limitations due to assumptions made during the whole procedure. Further recommendations and conclusion will be presented in the next chapter.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Recommendations from the study

Based on static analysis of wind loading, all existing structures are so far safe with respect to ‘right now’ condition of our region. However, getting ready with unforeseen circumstances of geological trends, enhancements should be implemented towards high rise moment resisting frame structures that are not designed by lateral loads especially with height beyond 77m. This can be done by providing additional bracing system between the columns. The total cost might be very high due to requirement of hacking off the wall of the structure for installation. However, for office buildings and hotels where renovations are expected for interior designing and other restorations, enhancements can be done to the interior column to reduce the effect of wind loadings

On the other hand, based on both studies of wind and earthquake loadings, the practical height of moment resisting frame without lateral loads design is restricted at 25 storey (87.5m). Any structure using moment resisting frames are no recommended to be beyond this height. For new structures, it is recommended to consider seismic loadings for the design of low rise shop-house or bungalows as they are theoretically more vulnerable to seismic compared to medium rise structures.

However, design of wind loading for low rise and medium rise is seems to be unnecessary as gravity load design is sufficiently optimised.

5.2 Recommendation to the study

Analysis using static method is considered as very conventional and conservative. It is just a very basic theory whereby the accuracy of the results is often questioned. Dynamic analysis method can be used as it includes the damping of the structures as well as the time factor of the loadings being imposed. It is always good to have comparison between the results of both static and dynamic analysis to see which one is more critical. However, based on discussion with supervisor, people agree that static analysis provide more exaggerated result compare to dynamic analysis. However, dynamic analysis is still considered as more convenient method in analysing behaviour of structure towards lateral loads.

To see this research in much more bigger yet deeper scope, experiments and lab works should be done to compare the results between theoretical method and practical method. Prediction from software analysis and practical observation from lab works are important in determining the variance between both approaches. The combination between theoretical teams and applied teams is critical so that more accurate findings can be achieved.

5.3 Conclusion

Based on analysis, conclusions are made with respect to objectives of study as follow;

I. Deflection due to wind loading is dependent on the ratio of exposed surface area to the number of columns
II. Deflection due to seismic loading is dependent on the ratio of total mass of the structure to the number of columns
III. As current condition in Malaysia, study shows that all structures are safe for 35 m/s wind load and medium rise structures are safe for ground acceleration

of 0.05g. These explain the zero documented structural failure so far due to these loads in Malaysia.

IV. However, all structures theoretically will be failed as magnitude of wind loadings and ground accelerations are slightly increased beyond typical condition on Malaysia. So, most of old structures without enhancement could possibly fail whenever geological conditions in Malaysia are getting worse than latest trends shown.

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