THE EFFECTIVENESS OF VARIOUS STRUCTURAL SYSTEMS IN REDUCING TALL BUILDING RESPONSE DUE TO WIND

ROSLIDA BT. ABD. SAMAT

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Universiti Teknologi Malaysia

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ABSTRACT

Recently, many tall building structural systems have been innovated in order to reduce the building responses due to wind loading. However, there are no systematic study conducted on the effectiveness of the different tall building systems in minimizing the responses of the building due to wind load. The objective of this research is to study the effectiveness of five tall building structural systems: core wall, outrigger, belt wall, tube-in-tube and megacolumns in minimizing the building responses due to wind. Reinforced concrete buildings with 64 stories and the ratio of height to the breadth of 6:1 were analysed for their responses to wind load. The buildings that were analysed have five different structural systems. The natural frequencies and eigenvectors of the buildings in the along-wind, across-wind and torsional mode are computed by a structural engineering software. The along-wind responses are determined by employing the procedures from the ASCE 7-02 while the across-wind and torsional responses of the buildings are calculated based on the procedures and wind tunnel data available in a data base of aerodynamic load. The database is comprised of high-frequency base balance measurements on a host of isolated tall building models. It is found that increasing the size of the core wall is more effective to reduce the building responses than increasing the thickness of the core wall. As for the outriggers, the most optimum position to construct the outriggers is between one quarter to two third of the height of the building. However, outrigger system is effective to reduce only the along-wind and across-wind responses. The torsional responses cannot be reduced by the addition of the outriggers. Interestingly, the addition of the belt walls will reduce the torsional response of the buildings which otherwise cannot be lessened by the outriggers. The belt walls also further reduce the building responses in the along-wind and across-wind directions. Moreover, the most optimal tube-in-tube structure is achieved when the spacing of the exterior columns is 4 metre, while the addition of megacolumns to the structural systems reduces the building responses drastically in all the three directions.

ABSTRAK

Kini banyak sistem struktur bangunan tinggi telah diperkenalkan bagi mengurangkan kelakunan bangunan terhadap beban angin. Namun begitu, tidak ada kajian yang sistematik dilakukan bagi menentukan keberkesanan sistem bangunan tinggi yang berbeza dalam mengurangkan kelakunan bangunan terhadap beban angin. Objektif penyelidikan ini adalah untuk mengkaji keberkesanan lima sistem struktur bangunan tinggi: dinding teras, rasuk sangga, dinding perimeter, tiub-dalam-tiub dan tiang mega bagi mengurangkan kelakunan bangunan akibat angin. Bangunan-bangunan konkrit bertetulang setinggi 64 tingkat dengan nisbah tinggi dan lebar 6:1 dikaji bagi mendapatkan kelakunan terhadap angin. Bangunan-bangunan yang dikaji ini mempunyai lima sistem struktur yang berbeza. Nilai frekuensi tabii dan eigenvektor dalam mod selari-angin, seranjang-angin dan puntiran bagi bangunan diperolehi dengan menggunakan perisian komputer kejuruteraan struktur. Kelakunan selari-angin ditentukan dengan menggunakan tatacara dari kod amalan ASCE 7-02, sementara kelakunan seranjang-angin dihitung dengan menggunakan tatacara dan data terowong angin dalam pengkalan data beban aerodinamik. Pengkalan data ini mengandungi bacaan alat imbangan asas berfrekuensi tinggi bagi model-model bangunan tinggi tunggal. Kajian menunjukkan penambahan saiz dinding teras adalah lebih berkesan untuk mengurangkan kelakunan bangunan dibandingkan dengan penambahan ketebalan dinding teras. Kedudukan paling optima untuk membina rasuk sangga pula ialah di antara satu perempat dan dua pertiga dari ketinggian bangunan. Namun begitu, rasuk sangga hanya berkesan untuk mengurangkan kelakunan selari-angin dan seranjang-angin. Kelakunan puntiran tidak boleh dikurangkan dengan penambahan rasuk sangga pada bangunan. Selanjutnya, penambahan dinding perimeter boleh mengurangkan kelakunan puntiran yang pada asalnya tidak boleh dikurangkan apabila rasuk sangga ditambah kepada sistem dinding teras. Selain itu, dinding perimeter juga boleh mengurangkan kelakunan bangunan selari-angin dan seranjang-angin. Struktur tiub-dalam-tiub yang optima boleh dicapai apabila jarak di antara tiang luaran adalah empat meter, manakala penambahan tiang mega adalah sangat berkesan dalam mengurangkan kelakunan bangunan pada semua tiga arah.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A	-	projected area of the structure loaded by the wind
a	-	length of the side of the square section
B,b	-	horizontal dimension of building measured normal to wind direction
C	-	coefficient of viscous damping
c	-	damping matrix,
c_c	-	critical damping
C_D	-	drag coefficient
C_f	-	mean along-wind force coefficient;
c_i^*	-	generalized damping in the i-th mode of vibration.
C_M	-	non-dimensional moment coefficient
d	-	height of zero-plane above the ground where the velocity is zero
D	-	dynamic matrix
E	-	modulus of elasticity
$\overline{E}$	-	**the load effect due to mean wind.**
$\mathbf{e}_i$	-	**error for each mode shape *i***
f	-	cyclic frequency
$\overline{f}_D(t)$	-	the mean parts of the drag force
$\tilde{\mathbf{f}}$	-	flexibility matrix
$\overline{f}_i$	-	mean generalized force
$f'_D(t)$	-	the fluctuating parts of the drag force
G	-	modulus of rigidity;

$\boldsymbol{G_f}$	-	gust factor
G_r	-	gradient velocity
G^{τ}	-	gust factor
G_q^{τ}	-	gust factor (GF) for wind velocity pressure.
$\overline{G}_{vz}$	-	gust velocity factor
$G_Y{}^T$	-	GLF for displacement
g, g_D	-	peak factor
$\boldsymbol{g_B}$	-	background peak factor
g_R	-	resonant peak factor
g_v	-	peak factor for upwind velocity fluctuations
H, h	-	average height of structure
$H(n)$	-	frequency response function
$\lvert H_1(f)\rvert^2$	-	structural transfer function of the first mode
I	-	moment of inertia
$\mathbf{I}$	-	N x N identity matrix and
I_h	-	turbulence intensity
$I(z)$	-	mass moment of inertia per unit height
I_H	-	turbulent intensity evaluated at the top of the building;
K	-	length to the power of fourth
$\mathbf{K}$, $[K]$, $\mathbf{k}$	-	stiffness matrix of the structure
K	-	surface drag coefficient
k_i^*	-	generalized stiffness in the i-th mode of vibration
k_T	-	torsional stiffness
L	-	horizontal dimension of a building measured parallel to the wind direction
L_u^x, L_u^y, L_u^z	-	integral scale
$\hat{M}_B$	-	background base moment and base torque
$\hat{M}_R$	-	resonant base moment or base torque response
$\overline{M}$	-	expected mean of the moment or torque response
$\overline{M}'$	-	reference moment or torque
$\hat{M}$	-	expected extreme value of the moment or torque response
m_i^*	-	generalized mass in the i-th mode of vibration.

m_1	-	modal mass
m	-	mass matrix,
m_{eff}	-	effective mass,
$\overline{m}$	-	mass per unit length along the beam
n_0, n_c	-	natural frequency of the structure in the across-wind direction
n_i	-	frequency in the *i*-th mode
P	-	load
$\hat{P}_B(z)$	-	equivalent static wind load for the background part
$\hat{\mathrm{P}}_R(z)$	-	resonant component of the equivalent static wind loading
$\hat{P}^*$	-	generalized force
$\overline{P}_1^*$	-	generalized load of the first mode;
$\overline{P}^\tau$	-	mean wind force with averaging time τ.
$\hat{P}^T(z)$	-	peak ESWL at height z during observation time T
p_i^*	-	generalized force in the *i*-th mode of vibration.
p	-	load matrix.
Q	-	non-dimensional quantities representing the normalized mean background responses
q_1	-	deflected shape
$q(z)$	-	mean wind velocity pressure
q_i	-	the *i*-th normal coordinate.
$\hat{q}_z$	-	peak dynamic pressure,
R	-	resonant response factor
$\hat{r}$	-	resultant wind-induced response of interest
$\overline{r}$, $\hat{r}_B$, $\hat{r}_R$	-	mean, peak beackground, and peak resonant response components
r	-	distance

S	-	Strouhal number
$S_M(f)$	-	power spectral density of force-balance-measured fluctuating base bending moment
$S_{p_i^*}$	-	spectral density of the generalized force
$S_P(z_1,z_2;f)$	-	cross spectral density of the aerodynamic load per unit height at z_1, z_2 and frequency f,
S_{qi}	-	power spectrum of the response
S_u	-	spectrum or spectral density of velocity
S_{u1u2}	-	cross spectrum of velocity
$S_v^*(f)$	-	normalized wind velocity spectrum with respect to the mean-square fluctuating wind velocity,
T	-	observation time
T_0	-	average time
T_{max}	-	maximum kinetic energy
t	-	time
$U(z)$	-	mean wind speed at z height
U_H	-	wind speed at the building height in the urban terrain
U_{max}	-	maximum potential energy,
U_t	-	wind speed averaged over t seconds
$\overline{U}$	-	mean wind speed at height z above the ground
$\overline{U}_{crit}$	-	critical wind speed
u, v, w	-	fluctuating components of the gust in x, y, z
u_*	-	shear velocity or friction velocity
V_{des}	-	maximum site wind speed multiplied by the importance factor
V_n	-	reduced velocity
$\overline{V}_{\bar{z}}$	-	mean hourly wind speed at height $\bar{z}$

$\hat{V}_{ref}$	-	3 s gust in exposure C at reference height
$\hat{V}_z$	-	peak wind velocity at height z
W_E	-	virtual work of the external force
W_i	-	virtual work of the internal force
δW_I	-	virtual work of the inertial forces per unit of length
X_{max}	-	maximum along-wind displacement
$\ddot{X}_{\max}$	-	maximum along-wind acceleration
x	-	mean displacement
$\ddot{x}$	-	mean acceleration
$\overline{Y}(z)$	-	the mean deflection
$\hat{\ddot{Y}}(z)$	-	peak along wind or across wind acceleration in the sway mode
Y	-	deflection
$\ddot{y}_2(x)$	-	accelerations would be developed along its length
$\hat{y}(z)$	-	maximum fluctuating deflection in the direction of the mean wind
$\mathbf{Z}$	-	amplitude
$\hat{\mathbf{Z}}^{(1)}$	-	first-cycle generalized-coordinate mode shapes and
Z	-	height above the surface,
z_{ref}	-	reference height normally taken to be 10 m
z_0	-	roughness length,
$\overline{z}$	-	reference height;
α	-	power law exponent
β	-	mode shape exponent.
$\beta\iota$	-	normalizing factor for iteration i
$\chi(n)$	-	aerodynamic admittance function
δ	-	gradient height

ε	–	error for all modes,.
$\hat{\mathbf{\Phi}}$	-	complete set of *N* normalized mode shapes
$\mathbf{\Phi}$	–	*N* modes of vibration matrix
ϕ	-	eigenvector matrix.
Λ , λ	-	diagonal matrix of the eigenvalues
θ	-	phase angle
ρ	-	air density
σ_{Bq_i}	-	non-resonating root mean square response of the *i*-th normal coordinate
σ_{Dq_i}	-	resonating root mean square response of the *i*-th normal coordinate
σ_M	-	root mean square (RMS) of the fluctuating base moment or base torque response
$\sigma_{\ddot{x}}(z)$	-	rms along-wind acceleration
$\sigma_y(z)$	-	root mean square value of the fluctuating deflection
τ	-	averaging time used to evaluate the mean wind velocity
Ω_1^2	-	first-cycle generalized-coordinate mode frequencies
ω	-	natural frequency of vibration.
ω_d	-	damped natural frequency
ω_i^2	-	eigenvalues
ξ	–	damping ratio
v	-	mean up-crossing rate
ψ	-	shapes of amplitude
ξ	-	mode exponent;
ζ_i	-	damping ratio in the *i*-th mode

LIST OF APPENDICES

CHAPTER 1

INTRODUCTION

1.1 Introduction

New structural systems including the composite one have allowed concrete high rises to reach new heights during the last four decades. There are a wide range of structural systems available for tall concrete buildings such as shear walls, core supported structures, tube in tube and bundled tubes.

During the design process, engineers must ensure the system is not only capable of resisting all loads, but also efficient, economic and satisfy the basic serviceability requirements. The design of tall building system is primarily dominated by the effects of wind. A tall flexible structure which is subjected to lateral or torsional deflections under the action of fluctuating wind loads may have oscillatory movements that can induce a wide range of responses in the building's occupants, ranging from mild discomfort to acute nausea. In fact, large displacements of these structures can cause improper drainage and damage of the windows and finishes of the building. Hence, the motions of the building that produce effects which is intolerable by the occupants may result in an otherwise acceptable structure becoming an undesirable or even unrentable building.

Therefore, it is important for engineers to compare a tall building response to wind forces with published data which describe on how the different values of the accelerations and displacements affecting human and the building itself. In order, to use these data, dynamic analysis is required to allow the predicted response of the building to be compared with the threshold limits.

1.2 Definition of Rigid and Flexible Building

A rigid structure is a structure which has the first few natural frequencies relatively high. The structure will tend to follow any fluctuating wind forces without appreciable amplification or attenuation. The dynamic deflections will not be significant, and the main design parameter to be considered is the maximum loading to which the structure will be subjected during its lifetime. Such a structure is termed "static" and it may be analyzed under the action of static equivalent wind forces (Stafford and Coull, 1991).

In contrast, a flexible structure has the first few natural frequencies relatively low. If the frequencies of the fluctuating wind are below the first natural frequency, the structure will tend to follow closely the fluctuating force actions. The dynamic response will be attenuated at frequencies above the natural frequency, but will be amplified at frequencies at or near the natural frequency. Consequently the dynamic deflections may be appreciably greater than the static values. The lateral deflection of the structure then becomes an important design parameter, and the structure is classified as "dynamic." Such structures require not only the dynamic stresses but also the acceleration induced by wind load to be determined during the design process (Stafford and Coull, 1991).

ASCE 7-02 defines flexible building or structure as slender building and other structures that have a fundamental natural frequency less than 1 Hz, while rigid building or other structures are defined as a building or other structure whose fundamental frequency is greater than or equal 1 Hz. The previous version of ASCE wind code, ASCE 7-98 also considers buildings that have a height, h, in excess of four times the least horizontal dimension as flexible buildings. As ASCE 7-02, both the Australian code (AS 1170.2) and Malaysian code (MS 1553:2002) also define a building as flexible or dynamic when its first fundamental natural frequency is less than 1 Hz

1.3 Drift Index and Acceleration Limit for Structures

Drift index is defined as the ratio of the maximum deflection at the top of the building to the total height. In addition, the corresponding value for a single story height, the inter-story drift index, gives a measure of possible localized excessive deformation. Balendra (1993) describes the effects of excessive deflection on building component in Table 1.1.

Table 1.1: Serviceability problems at various deflection or drift indices (Balendra, 1993)

Deformation as a fraction of span or height	Visibility of deformation	Typical behaviour
1/500	Not visible	Cracking of partition walls
1/300	Visible	General architectural damage Cracking in reinforced walls Cracking in secondary members Damage to ceiling and flooring Facade damage Cladding leakage Visual annoyance
1/200 – 1/300	Visible	Improper drainage
1/100 – 1/200	Visible	Damage to lightweight partitions, windows, finishes Impaired operation of removable components such as doors, windows, sliding partition

Design drift index limits that have been used in different countries range from 0.001 to 0.005. Generally, lower values should be used for hotels or apartment buildings than for office buildings, since noise and movement tend to be more disturbing in the former. Sound engineering judgment is required when deciding on the drift index limit to be imposed. However, for conventional structures the preferred acceptable range is 0.0015 to 0.003 (that is, approximately 1/650 to 1/350). As the height of the building increases, drift index coefficients should be decreased to the lower end of the range to keep the top story deflection to a suitably low level.

The National Building Code of Canada (1990) limits the drift to 1/500 of the height in order to limit the cracking of the masonary and interior finishes unless detailed analysis is made and precautions are taken to permit larger movements. Malaysian code (MS 1553:2002) also limits the total drift of wind force resisting system to 1/500 of the height, and the inter-story drift to 1/750 of the height.

Furthermore, Clause B.1 .2 in the ASCE7-02 requires the lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects not to impair the serviceability of the structure. However, Clause CB.1.2 in the ASCE7-02 states that the drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control, 1988). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. Clause CB.1.2 in the ASCE7-02 also indicates that an absolute limit on inter-story drift may also need to be imposed in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the inter-story drift exceeds about 10 mm (3/8 in) unless special detailing practices are made to tolerate movement (Freeman 1977; Cooney and King 1988). However, many components can accept deformations that are significantly larger.

There are as yet no generally accepted international standards for comfort criteria, although they are under active consideration. In recent years, a considerable amount of research has been carried out into the important physiological and psychological parameters that affect human perception to motion and vibration in the low frequency range of 0-1 Hz encountered in tall buildings. It is now generally agreed that acceleration is the predominant parameter in determining the nature of human response to vibration (Irwin, 1986). Table 1.2 and Table 1.3 illustrate how human behaviour and motion perception are affected by different ranges of acceleration.

Table 1.2: Human perception level (Yamada and Goto,1975)

Range	Acceleration (m/sec^2)	Effect
1	< 0.05	Humans cannot perceive motion
2	0.05 – 0.10	Sensitive people can perceive motion; hanging objects may move slightly
3	0.1 – 0.25	Majority of people will perceive motion; level of motion may affect desk work; long-term exposure may produce motion sickness
4	0.25 – 0.4	Desk work becomes difficult or almost impossible; ambulation still possible
5	0.4 – 0.5	People strongly perceive motion; difficult to walk naturally; standing people may lose balance
6	0.5 – 0.6	Most people cannot tolerate motion and are unable to walk naturally
7	0.6 –0.7	People cannot walk or tolerate motion
8	> 0.85	Objects begin to fall and people may be injured

Table 1.3: Acceleration limits for different perception level (Balendra, 1993)

Perception	Acceleration Limit
Imperceptible	$a < 0.005g$
Perceptible	$0.005g < a < 0.015g$
Annoying	$0.015g < a < 0.05g$
Very annoying	$0.05g < a < 0.15g$
Intolerable	$a > 0.15g$

In order to check the serviceability of tall buildings, the along wind, across wind and torsional responses are determined individually before combining them vectorally. A reduction factor of 0.8 may be used on the combined value to account for the fact that in general the individual peaks do not occur simultaneously. If the calculated combined effect is less than any of the individual effects, then the latter should be considered for the designs (Balendra, 1993).

Since the tolerable acceleration levels increase with period of building, the recommended design standard for peak acceleration for 10-year wind in commercial and residential buildings is as depicted in Figure 1.1 (Griffis,1993). Lower acceleration levels are used for residential buildings for the following reasons:

1. Residential buildings are occupied for longer hours of the day and night and are therefore more likely to experience the design wind storm
2. People are less sensitive to motion when they are occupied with their work than when they relax at home.

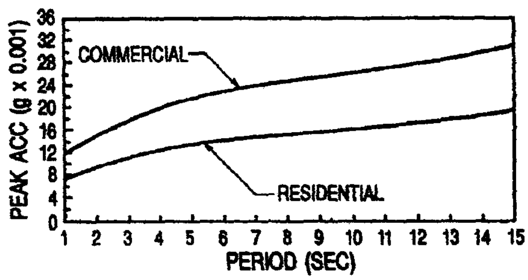

Figure 1.1: Design standard on peak acceleration for 10-year return period (after, Griffis, 1993)

The National Building Code of Canada (1990) recommends the acceleration limit to be 1-3% of gravity (0.09 to 0.27 m/sec^2) once in every 10 years, the two figures being more appropriate for apartment and office blocks respectively. Malaysian wind code , MS 1553:2002 requires the acceleration of a building due to wind-induced motion not exceed 1.0% of gravity for residential structures and 1.5% of gravity for other structures, of the acceleration due to gravity.

Clause CB1.3 in the ASCE7-02 states that excessive structural motion is mitigated by measures that limit building or floor accelerations to levels that are not disturbing to the occupants or do not damage service equipment. Perception and tolerance of individuals to vibration is dependent on their expectation of building performance (related to building occupancy) and to their level of activity at the time the vibration occurs (ANSI 1983). Individuals find continuous vibrations more objectionable than transient vibrations. Continuous vibrations (over a period of minutes) with acceleration on the order of 0.005 g to 0.01 g are annoying to most people engaged in quiet activities, whereas those engaged in physical activities or spectator events may tolerate steady-state acceleration in the order of 0.02g to 0.05g. Thresholds of annoyance for transient vibrations (lasting only a few seconds) are considerable higher and depend on the amount of structural damping present (Murray, 1991). A typical finished floor will have 5% damping or more and peak transient accelerations of 0.05 g to 0.1 g may be tolerated.

1.4 Problem Statement

Despite the importance of analyzing the building responses (displacement and acceleration) as explained in Section 1.3, there are no systematic study that has been conducted on the effectiveness of the different tall building systems in minimizing the responses of the building due to wind. There are several tall building systems available such as outriggers, belt wall, tube-in-tube, core wall and mega columns. However, no study has been performed to determine on how effective these tall building systems are in reducing the displacements and accelerations of tall buildings that are being exerted by wind forces. It is not known which tall building system is the most effective system to reduce the responses of the buildings due to wind.

Research on the effect of certain parameters such as dimension and location of the structural systems in the effectiveness of the systems in reducing the building responses has also not been performed. Is increasing the thickness of the core wall or increasing the dimension of the core wall is better in reducing the responses of the building due to wind? Where is the best location to place the outriggers so that the responses of the building due to wind can be minimized? How effective is the belt wall in reducing the responses of the building due to wind compared to outrigger system? What is the optimum spacing of the parameter columns of the tube-in-tube systems in reducing the responses of the buildings due to wind? How effective is megacolumn system in minimizing the responses of the buildings due to wind?

1.5 Objective

The objective of this research is to study the effectiveness of five tall building structural systems: core wall, outrigger, belt wall, tube-in-tube and mega column in minimizing the building response (displacement and acceleration) to wind. There are different objectives to be accomplished for each different tall building structural studied. The objective of studying :

- the core wall is to determine whether increasing the thickness or increasing the dimensions of the core wall is more effective in reducing the responses of the building due to wind;
- the outrigger is to determine the best location to construct the outrigger so that the responses of the building due to wind can be minimized;
- the belt wall is to study the effectiveness of the belt wall in reducing the responses of the building due to wind compared to the outrigger system;
- the tube-in-tube system is to find the optimal spacing of the perimeter columns in minimizing the responses of the building due to wind;
- the megacolumn system is to study the effectiveness of this system and combination of megacolumn and other structural elements such as outriggers and belt wall in reducing the responses of the building due to wind.

Another objective of this research is to determine which tall building system among the five systems: core wall, outriggers, belt wall, tube-in-tube and mega columns studied that is the most effective system in minimizing the responses of the building due to wind.

1.6 Scope

The building studied is a tall flexible building which has a square plan. A tall flexible building must have the ratio of height to the lateral dimension more than 1:4 and natural frequency less than 1 Hz as explained in Section 1.2. Thus, the buildings studied has the ratio of height to the lateral dimension of the building 1:6 and their natural frequencies will be less than 1 Hz. The reason of choosing the ratio of height to the lateral dimension of the building 1:6 is because the ratio 1:6 is the largest ratio of height to the lateral dimension of the building available in the aerodynamic data base. It is impossible to obtain the values of the across-wind and torsional responses for buildings if the ratio of height to the lateral dimension of the building is more than 1:6 as experimental data for these buildings are not available. Note that currently, no formula is available in calculating the across-wind responses and torsional responses. The formulae that are available such as from the Australian Code (AS 1170.80), Japanese code (RLB-AIJ-1993), Canadian code (NBC-1995), the aerodynamic data base of University of Notre Dame, United States of America and other literatures are empirical formulae that are based on experimental data(Simiu and Scanlan, 1996).

The lateral system of the tall building is reinforced concrete. Five types of tall building systems will be analyzed are:

- Core wall
- Outrigger
- Outrigger with belt wall
- Tube-in-tube
- Megacolumn

The manipulated variables for each system are presented in detail in Chapter 3.

The building will be exerted by wind loading for three different wind environments which are:

- Malaysia (benign wind environment)
- New York (aggressive wind environment)
- Hong Kong (one of the most aggressive wind environment in the world)

According to Holmes (2003), the extreme wind classification for Malaysia, New York and Hong Kong is I, III and IV, respectively. Holmes has developed classification systems to 'grade' any country or region in terms of its general level of wind speed. Level I has the lowest wind speed while Level V has the highest wind speed. Chapter 3 will describe further, about the values of wind speed used and the calculation of wind speed for different averaging time.

1.7 Methodology

The methodology of this project is as shown in the flowchart in Figure 1.2, and is described in detail in Chapter 3.

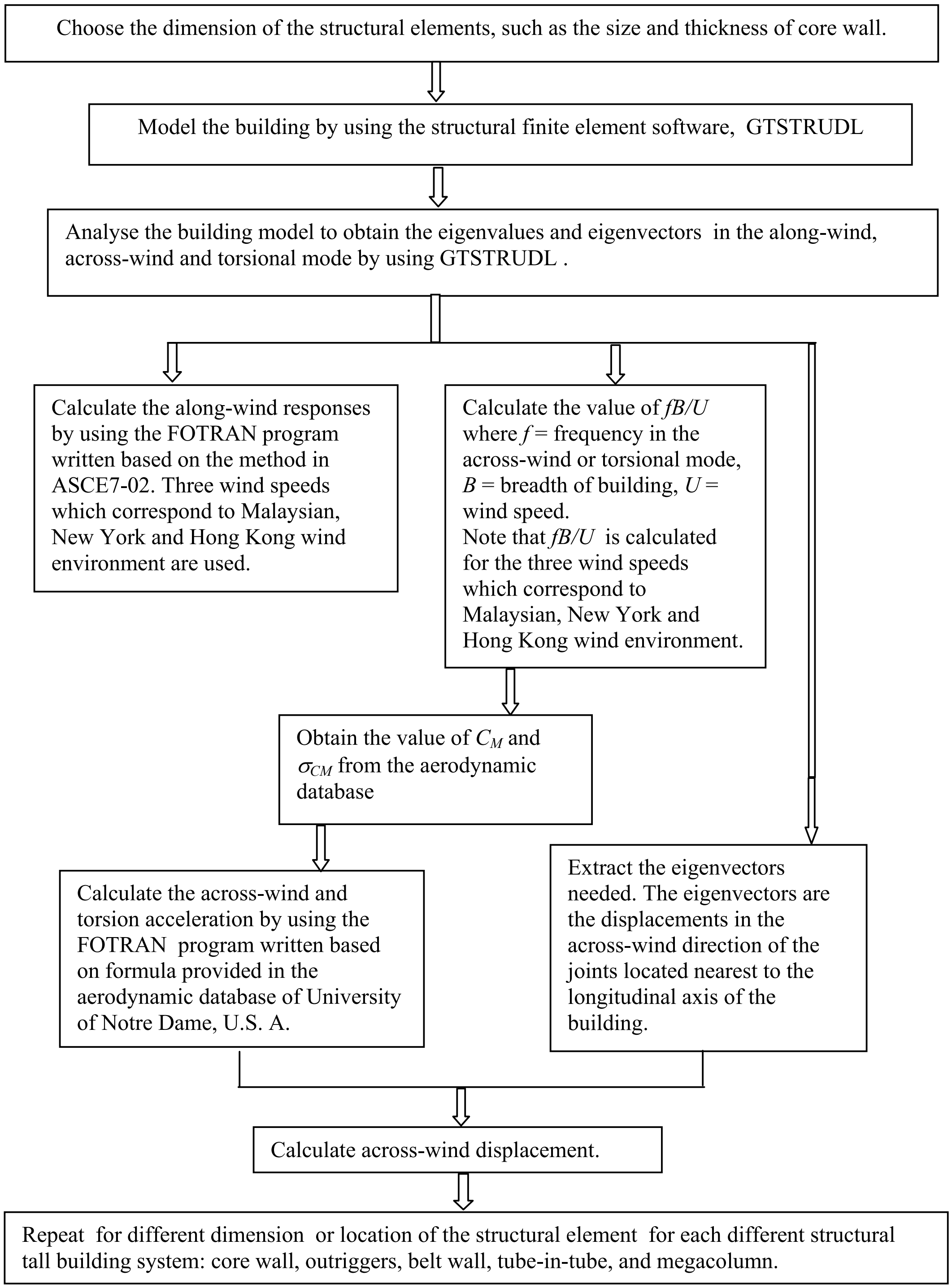

Figure 1.2: A flow chart showing the steps taken in the calculation of responses of the buildings due to wind.

1.8 Overview of the Thesis

Chapter 2 is a literature review. It will discuss about the development of the research of the effects of wind to tall building. It also explains briefly about the important subjects in wind engineering such as averaging wind speed, wind profile, along-wind, across-wind and torsional response of tall buildings.

Chapter 3 will describe about the methodology used in the research in depth. Not only will it discuss on how the building is modeled in order to obtain the natural frequency and eigenvector, but it will also discuss about the problems in performing eigenvalue analysis by using the finite element methods. This chapter will also describe the procedure to determine the along-wind responses in ASCE 7-02 and across-wind and torsional responses in the University of Notre Dame aerodynamic database.

Chapter 4 will present the results obtained from the study for three tall building systems: core wall, outriggers and belt wall systems. These results are discussed in detail in this chapter.

Chapter 5 will provide results from the study for the other two building systems: tube-in-tube and megacolumns. The results are also discussed in depth in this chapter.

Finally, Chapter 6 will draw the conclusions on this project. Suggestions for further study in this area will also be proposed.

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