Seismic Analysis on Multi-Storey Steel Structures in Malaysia

by

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CERTIFICATION OF APPROVAL

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Civil Engineering Programme
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Approved by,

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Assoc. Prof. Dr. Narayanan Sambu Potty

UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
May 2013

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 SYAFIQ BIN YUNUS

ABSTRACT

The purpose of this study is to investigate the effect of earthquake in low, medium and high rise structures in Malaysia. Structural design in Malaysia may overlook the significance of earthquake loading as earthquakes rarely happen in Malaysia region. The occurrences of several tremors in neighboring countries such as Philippines and Indonesia have triggered a series of vibrations which were felt on some of the buildings in Malaysia. This study shows the analysis of low, medium and high rise steel structures subjected to earthquake loading in provision of Eurocode 8 EN 1998-1:2004.

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CHAPTER 1

PROJECT BACKGROUND

1.1. BACKGROUND OF STUDY

The issues of Malaysia’s safety on earthquake were raised by the public recently. Tremors in Peninsular Malaysia and East Malaysia due to Sumatra and Philippine earthquakes have been reported several times. Engineers are concerned of the seismic vulnerability of public buildings due to lack of earthquake consideration in Malaysia’s building design procedure (Rozaina et al., 2011). National Geographic reported that seismic activity in the earth’s crust happened every day in varying degrees of intensity and Malaysia, although not in the Pacific Ring of Fire danger zone, the danger still pose a significant threat. Malaysia may face medium earthquakes, which is strong enough to damage buildings nationwide. New Straits Times reported that the most powerful earthquake ever recorded in the country so far was of a medium-intensity measuring 5.8 on the Richter scale in Lahad Datu, Sabah in 1976 which resulted in damage to property and buildings. In the interview by Selvarani (2012), Malaysian Meteorological Department Geophysics and Tsunami division director Dr Mohd Rosaidi Che Abas reminded that while Malaysia is not in the active subduction zone, it remained at risk if strong earthquakes, such as the one which hit Aceh on Dec 26, 2004, occurred anywhere along the western coasts of Sumatra and the Philippines. However, the construction of major reservoirs and dams, or the pumping of pollutants deep in the subsurface, can modify the stress and strain on the earth’s crust, this induce seismicity which can cause minor earthquakes and tremors.

1.2. PROBLEM STATEMENT

Malaysia has been affected seismically by far field earthquakes events from neighboring countries since years back. Such matter should be factored in when designing structures. Most of the earthquakes 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 more dense area with lots of superstructures and high rise building like in Kuala Lumpur? Azlan & Meldi (2009) stated that the nearest distance of earthquake epicenter from Malaysia is approximately 350 km. Natural phenomenon like earthquake causes damage to or collapse of buildings if not designed for lateral loads resulting due to Earthquake (Ventakesh et al., 2012). Hence, there are problems are raised in this seismic hazard analysis. This study is going to be vital in order to answer the question by assessing high rise structure in term of their structural integrity in facing earthquake and hurricane. We are going to evaluate how far our structure can withstand in various modes of shaking conditions concurrent with the seismic condition in Malaysia.

1.3. OBJECTIVE

The objectives of this study are as the following:

i. To study the behavior of low, medium and high rise structure through determining natural frequency
ii. To determine the multi-storey drift of the building on passing traditional or conventional Malaysia design.
iii. To redesign such structure to earthquake loading and reevaluate the multi-storey drifts.

1.4. SCOPE OF STUDY

This study focus on the behavior of high-rise steel structures as designed by dead load, live load, and wind load to the additionally earthquake loads. The analysis will involve only bare frame of the structure without considering the effect on its infill. This study will only cover the height variation of buildings and its reaction towards possible earthquake in Malaysia. Although there are no visible and physical structural failures due to these loads in Malaysia, this analysis will observe the horizontal displacement in its serviceability limit states. Equivalent Static Analysis and Dynamic Analysis will be used to assess the building response due to earthquake loading. Simulation and analysis is done by using computer software such as StaadPro. 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.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

There are six (6) sub-sections in this chapter that are going to enlighten the readers regarding the study. First sub-section titled 'Earthquake Hazard in Malaysia' generally tells the readers about series of earthquake in Malaysia and its severances while the next part, 'Building Response to Earthquake Loading' will give the ideas of how building will react to earthquake loading. Later, 'Earthquake Design Analysis' will elaborate on the several techniques of analysis that can be applied to multistory structures. 'Seismic Design Philosophy' will explain the severity of the earthquake against the overall damage to the building. 'Use of Computer Aided Design in Analysis' will discuss on tools available to be used in analyzing structural behavior.

2.2 EARTHQUAKE HAZARD IN MALAYSIA

Although Malaysia are located on a stable part of the Eurasian Plate, buildings on soft soil are occasionally subjected to tremors due to far-field effects of earthquakes in Sumatra (Balendra et al. 1990). In the last few years, tremors were felt several times in tall buildings in Kuala Lumpur, the capital of Malaysia, due to large earthquakes in Sumatra. Although situated on the stable shelf, several places especially in Northern Peninsular Malaysia, which is Penang Islands, Alor Star and Ipoh have experienced ground shaking effect due to the long distant earthquake occurred in Acheh and Nias recently. Northern Penang are situated close to the earthquake tremors may demand a quick review on the existing design code for designing structures (Taksiah et al., 2007). The high frequency earthquake waves damped out rapidly in the propagation while the low frequency or long period waves are more robust to energy dissipation and as a result they travel long distances. Thus the seismic waves reaching the bedrock of Malaysia are rich in long period waves, and are significantly amplified due to resonance when they propagate upward through the soft soil sites with a period close to the predominant period of the seismic waves. The amplified waves cause resonance in buildings with a natural period close to the period of the site, and the resulting motions of buildings are large enough to be felt by the residence (Balendra and Li, 2008). The recent high intensity earthquakes in 2004 and 2005 from Sumatra, Indonesia have severely jolted the population of Peninsular Malaysia with appreciable ground movements. Bing & Tso (2004) mentioned that BS code 8110 used in Malaysia doesn't not specify any requirement for seismic design or detailing of structures. These have raised questions on the structural stability and integrity of existing building structures in Malaysia, in the face of such seismic effect from Sumatra – which is termed as "Far Field Effect" of earthquake (Jeffrey, 2008).

2.3 BUILDING RESPONSE ON EARTHQUAKE LOADING

Buildings respond significantly when they are shaken at frequencies close to their natural frequency. Hugo (2003) pointed out that if the ground moves rapidly back and forth, then the foundations of the building are forced to follow these movements. The upper part of the building however would prefer to remain where it is because of its mass of inertia. Azadbakht & Barghi (2009) state the natural frequency or period can be estimated using building design codes. The response of a building to an earthquake underneath it is different than that due to wind blowing on it. Like all physical systems, buildings also respond to earthquake shaking through its modes of vibration. So as long as buildings behave linearly, these modes of vibration are easy to ascertain, because they are constant throughout the shaking; but, when buildings go into nonlinear behaviour (in general, nonlinearity in buildings is of softening-type), the modes of vibrations constantly keep changing (Murty, 2006). The effect of infill panel structure subjected to seismic action is widely recognized and has been subject of numerous experimental investigations. Wakchaure & Ped (2012) found out that infill walls reduce displacements, time period and increase base shear. Diware & Saoji (2012) agreed that infills frame have greater strength as compared to frames without infills. Although infills have significant effect on the result output, the author only cover only for bare frame without considered the effect of infills for preliminary study.

2.4 EARTHQUAKE DESIGN ANALYSIS

Alsulaydani and Saaed (2009) pointed out that many methods are available for the structural analysis of buildings and other civil engineering structures under seismic actions. Abu (2010) stated that Seismic design of buildings depends on peak ground acceleration values and shape of Response Spectra curves as depicted by relevant Building codes. Carlos (2006) demonstrated 2 methods of seismic design of multistory structures; Equivalent Static Force and Dynamic Analysis which can take a number of forms. Mode superposition is one of these forms. Behaviour of buildings under dynamic forces depends upon the dynamic characteristics of buildings which are controlled by both their mass and stiffness properties, whereas the static behaviour is solely dependent upon the stiffness characteristics (Hemant et al., 2006). Sinadinovski et al. (2005) observed that dynamic analysis can provide more accurate distribution of the lateral load. The methods of dynamic analysis used are Time History Method and Response Spectrum Method. Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non linear. Time history analysis is used to determine the dynamic response of a structure to arbitrary loading. A response spectrum may be visualized as a graphical representation of the dynamic response of a series of progressively longer cantilever pendulums with increasing natural periods subjected to a common lateral seismic motion of the base (Mohan et al., 2011)

Amit (2012) highlights the importance of explicitly recognizing the presence of the open storey in the analysis of the building if there is any. Infll walls, however, are treated as non-structural components even though they provide significant improvement in lateral stiffness of the frame structures. (Jigme, 2009). Experience from the past earthquakes show that strong infill, although non engineered, often provide most of the lateral resistance and prevent collapse of relatively flexible and weak reinforced concrete frames that are necessarily not designed for the seismic forces (Sujatha et al., 2009). The error involved in modeling such buildings as complete bare frames, neglecting the presence of infills in the storeys, is brought out through the study of an example building with different analytical models. According to Edward (2002) the mode displacement superposition method provides and efficient means of evaluating the dynamic response of most structures because the response analysis is performed only for a series of SDOF systems. The response analysis for the individual modal equations requires very little computational effort and in most cases only a relatively small number of the lowest modes of vibration need to be included in the superposition. The basic mode superposition method, which is restricted to linearly elastic analysis, produces the complete time history, response of joint displacements and member forces. Acceleration Time-histories of earthquake ground motions are required for analyzing the structural performances and response of soil deposits under seismic loading. Selection of appropriate time-histories for specific geological and seismological conditions plays an important role for obtaining accurate results. (Azlan, Hendriyawan, Amination, Masyur, 2006). Andreas & Georgios (2004) propose time-history analysis as a tool of analysis.

2.5 SEISMIC DESIGN PHILOSOPHY

Edgar & Mark (2000) and Pankaj & Manish (2006) mentioned that the design philosophy is to ensure that the structures possess at least a minimum strength to; resist minor earthquake without damage, resist moderate earthquake (Design Basis Earthquake) without significant structural damage though some non-structural damage and resist major earthquake (Maximum Considered Earthquake) without collapse. According to Indian Standard (2002), Design Basis Earthquake (DBE) is defined as the maximum earthquake that reasonably can be expected to experience at the site during lifetime of the structure. The earthquake corresponding to the ultimate safety requirement is often called as Maximum Considered Earthquake (MCE). Generally, the DBE is half of MCE. The basic criteria of earthquake resistant design should be based on lateral strength as well as deformability and ductility capacity of structure with limited damage. As stated in the Eurocode, the interstorey drift limitation for buildings having non-structural elements of brittle materials attached to the structure would be Storey Height/200.

2.6 USE OF COMPUTER AIDED DESIGN IN ANALYSIS

Pankaj & Manish (2006) state the procedure dynamic analysis of buildings may be based on 3D modeling of building. According to Bedabrata & Nagender (2007), STAAD.Pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.Pro is essential choice of computer aided design software for this project. Software ETABS may also used in this project as demonstrated Wakchaure (2012) in his earthquake analysis of high rise buildings. It has been proved by more and more practices that the simulation technique (ST) can get a more satisfied result than experiments in some cases like large-span or high-raise structures (Zhao et al., 2012).

2.7 SUMMARY

Based on thorough review from various journals and other types of literature, it is understood that the study on effects of lateral loading especially is very important to ensure that structures are prepared to face unforeseen and undesirable circumstances such as natural disaster. Buildings are built along with their integrity and consistency and it is very important to ensure that they can serve their functions and purposes without any major problem. Most part of the literature discussed on the behaviour of the frame structures as being imposed by lateral load. This is crucial to this study in order to check the possible failure modes and their adverse effects so that analysis could be done with the most accurate judgements and assumptions. It is important for the modelling to be as related as possible to the real condition and trending. In conclusions, these literature reviews are very critical and influential for this analysis of existing high rise steel structure in Malaysia with subject to earthquake loadings.

CHAPTER 3

METHODOLOGY

3.1 INTRODUCTION

The idea behind the following methodology is to compare the actual storey drift obtained against the allowable one stated in the Eurocode. There are two analysis involved in this report; Equivalent Static Analysis and Dynamic Analysis. The dynamic analysis will involve the appropriate selection of ground motion, apply to the structure and analyze it. Figure 1 shows the flow chart of methodology of this analysis. The details of the methodology will be explained further later in this chapter.

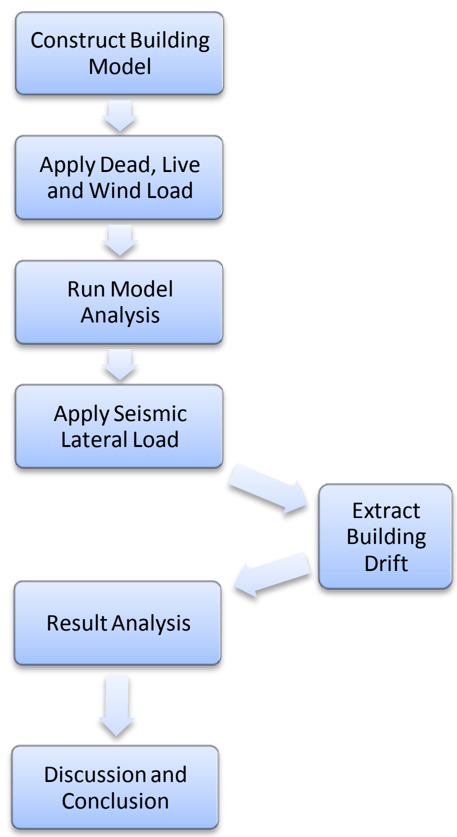

Figure 1 Methodology Flow Chart

3.2 SELECTION OF DESIGN RESPONSE SPECTRA

In this project, we are focusing on using Response Spectrum Method in Dynamic Analysis to evaluate the drifting of multistory building due to earthquake loading. Response spectra method involve the determination of Eigenvalues and Eigenvectors based on the mass and stiffness of the structures, modal participation factors, modal mass. From the previous calculation, we would be able to determine the lateral force at each floor and corresponding storey shear forces in each mode. The peak storey shear force in particular storey due to all modes considered is obtained by combining those due to each mode in accordance with modal combination such as SRSS (Square Root of Sum of Squares) or CQC (Complete Quadratic Combination) methods. However, the respond spectra method can only calculated manually up to 3 storeys only due to tedious and long process of calculation. The design response spectra (Fadzli, 2007) used for this study is shown in Figure 2. The details of the value of response spectrum acceleration against period are shown in Table 1.

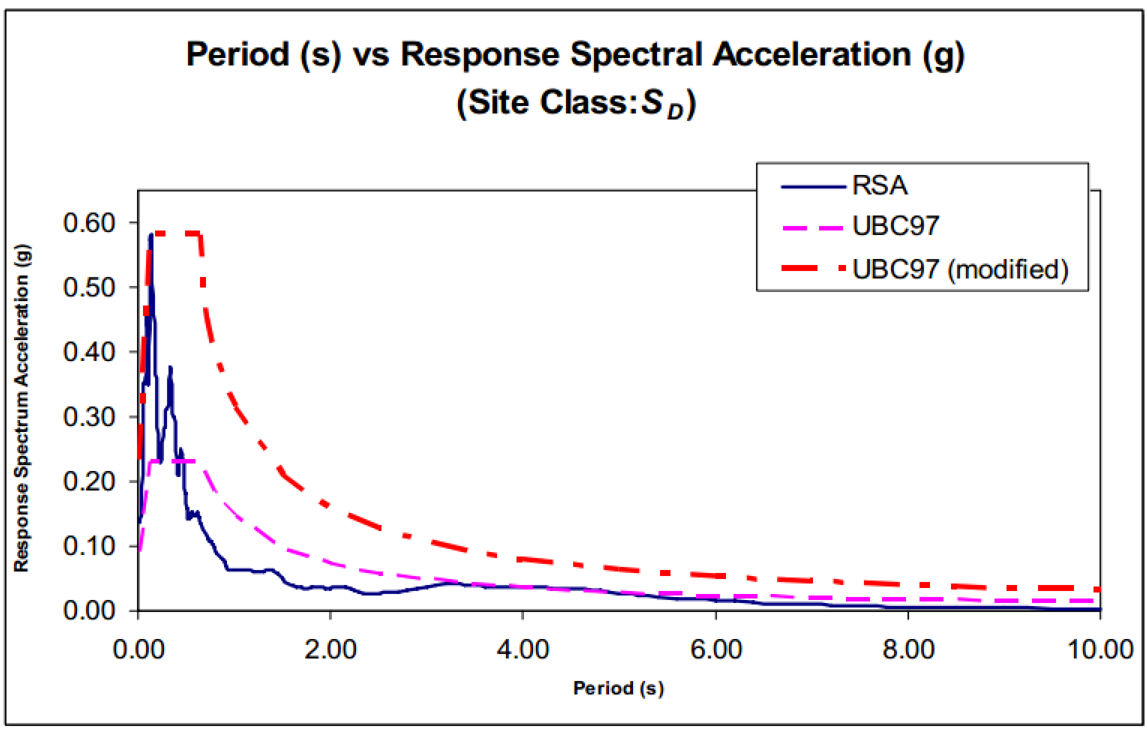

Figure 2 Design Response Spectrum (Taksiah, A. M., Shaharudin, S. Z., Fadzli, M.N., Mohd, R. A. & Izatil, F. M. S. (2007). Development of Design Response Spectra For Northern Peninsular Malaysia Based on UBC 97 Code. School of Civil Engineering, Universiti Sains Malaysia)

Period (s)	Modified RSA (g)
0.01	0.2331
0.1271	0.5828
0.6356	0.5828
0.70	0.4500
0.80	0.3938
0.90	0.3500
1.00	0.3150
1.50	0.2100
2.00	0.1575
2.50	0.1260
3.00	0.1050
3.50	0.0900
4.00	0.0788
4.50	0.0700
5.00	0.0630
5.50	0.0573
6.00	0.0525
6.50	0.0485
7.00	0.0450
7.50	0.0420
8.00	0.0394
8.50	0.0371
9.00	0.0350
9.50	0.0332
10.00	0.0315

Table 1 Response Spectrum Value

3.3 CONSTRUCT BUILDING MATERIAL IN STAADPRO

3.3.1 General

There are several multi-storey buildings involved which have height variation; 3, 5, 10, 20 & 30 storey buildings. They are modeled in StaadPro 2004 and checked against British Code BS 5950 after all the loads and member properties are assigned. The details of loadings and property materials will be explained further in the later section. The building is constructed in a way that it has column to column distance of 6 meter, on both x-axis and z-axis. The slab thickness is taken as 150 mm and made up of concrete with inclusive of finishes. The storey height will be as 3 meter as it is common practice in Malaysia and also stated in Malaysia Law that it is the minimum required storey height of buildings. The buildings will be treated as residential buildings due to huge number of multistory buildings are built for residential purpose for increasing population in city centre. The building is made up of steel frame material as stated in scope. The design of the beam and column are made typical as much as possible for optimization and ease of analysis.

3.3.2 Loadings

The loads involved in the building design are mainly consisting of Dead Load, Live Load and Wind Load. The load combination also will follow as stated in the British Code. The design loads for buildings and other structures shall be as specified in BS 6399 except as specified herein.

Dead Load

Dead load is the load due to self weight of the structure, the weight of all walls, permanent partitions, floors, roofs, finishes and all other permanent construction including services of a permanent nature. The brick wall load will be assigned to the entire beam in between storey. The unit weight of structure proper shall be as follows:

Reinforced Concrete	:	24.0 kN/m^3
Plain Concrete	:	23.0 kN/m^3
Steel	:	77.0 kN/m^3
Full Brick wall	:	5.2 kN/m^2

The details of Dead Load applied on the structure will be as followed:

a)	RC Slab	= 24 kN/m^3 x 0.15m		= 3.6 kN/m^2
b)	Finishes	= 24 kN/m^3 x 0.05m		= 1.2 kN/m^2
			Total	= 4.8 kN/m^2
c)	Brick wall	=5.2 kN/m^2 x 3.0 m		= 15.6 kN/m

Live Load

Live Load is the load assumed to be produced by the intended occupancy or use, including the weight of movable partitions, distributed, concentrated, impact and inertia, loads, but excluding wind loads. The value of 2 kN/m^2 will be used as live load according to BS 6399 and applied from first floor until roof floor. Reduction in total distributed imposed floor loads shall be in accordance with table 2 & 3 of BS 6399 Part 1. This reduction is necessary because the resulted loads on the ground floor column will be very tremendous and not design optimized if it is not considered.

Number of floors including the roof	Reduction (%)
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

Table 2 Reduction in Total Distributed Imposed Floor Loads with Number of Storey

Area supported (m2)	Reduction (%)
0	0
50	5
100	10
150	15
200	20
Above 250	25

Table 3 Reduction in Total Distributed Imposed Floor Loads on A Supporting Beam Or Girder With Floor Area

Wind Load

The wind load design as per Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions.

Determination of basic wind velocity:

$$v_b = c_{dir} \times c_{season} \times v_{b,0}$$

Where:

v_b	basic wind velocity
c_{dir}	directional factor
c_{season}	seasonal factor
$v_{b,0}$	fundamental value of the basic wind velocity

The fundamental value of the basic wind velocity, vb,0, is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain. Malaysian Standard, MS 1553: 2002 provide the value of 33.5 m/s for 3 second gust wind speed when translated into 10 minutes mean wind speed is:

$$v_{b,0} = 24 \text{ m/s}$$

Since the area of study is located in Kuala Lumpur, the selection for terrain categories and terrain parameters will be the area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m:

Terrain category IV $\Rightarrow$ $z_0 = 1$ m

$z_{min} = 10$ m

$z_{max} = 200$ m

z = Building Height (in meter)

So the basic wind velocity will be:

$$v_b = c_{dir} \times c_{season} \times v_{b,0} = 24 \text{ m/s}$$

For simplification the directional factor c_{dir} and the seasonal factor c_{season} are in general equal to 1.0.

The peak velocity pressure, $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, should be determined.

$$q_p(\mathrm{z}) = \underbrace{\left[1 + \frac{7k_I}{c_o(z)\times\ln(z/z_0)}\right]}_{squared\ gust\ factor} \times \underbrace{\frac{1}{2}\times\rho\times v_b{}^2}_{basic\ pressure} \times \underbrace{\left(k_T\times\ln(z/z_0)\right)}_{wind\ profile}$$

Where: k_I is the turbulence factor (Recommended value for k_I is 1.0)

$c_o(z)$ is the orography factor (1.0)

ρ is the air density (1.25 kg/m^3)

k_T is the terrain factor, depending on the roughness length z0 calculated using

$$k_T = 0{,}19\times\left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}$$

Where: $z_{0,II}$ = 0,05 (terrain category II)

Thus, k_T = 0.234

Internal and external pressures are considered to act at the same time. The wind loadings per unit length w (in kN/m^2) are calculated:

$w = (c_{pe} + c_{pi}) \times q_p$

Where c_{pe} is the pressure coefficient for the external pressure depending on the size of the loaded area A. (equal to $c_{pe,10}$ because the loaded area A, 36m^2 for the structure is larger than 10 m^2). For all the height/width is more than 1 and less than 5, $c_{pe,10}$ is equal to +0.8).

The internal pressure coefficient, c_{pi} depends on the size and distribution of the openings in the building envelope. Within this study, it is not possible to estimate the permeability and opening ratio of the building. So c_{pi} should be taken as the more onerous of + 0.2 and – 0.3. In this case cpi is unfavorable when cpi is taken to + 0.2.

Thus, $w = (c_{pe} + c_{pi}) \times q_p = (0.8+0.2) \times q_p = q_p$

The details of wind load can be shown in Table 4.

No of Storey	Building Height, z	Peak Pressure, q_p (kN/m2)	Wind Load, w (kN/m2)
3	9	0.775	0.775
5	15	0.818	0.818
10	30	0.876	0.876
20	60	0.935	0.935
30	90	0.969	0.969

Table 4 The Details of Wind Loads to Respective Buildings

Below is the screenshot how the wind loads are applied to the 20 storey in StaadPro.

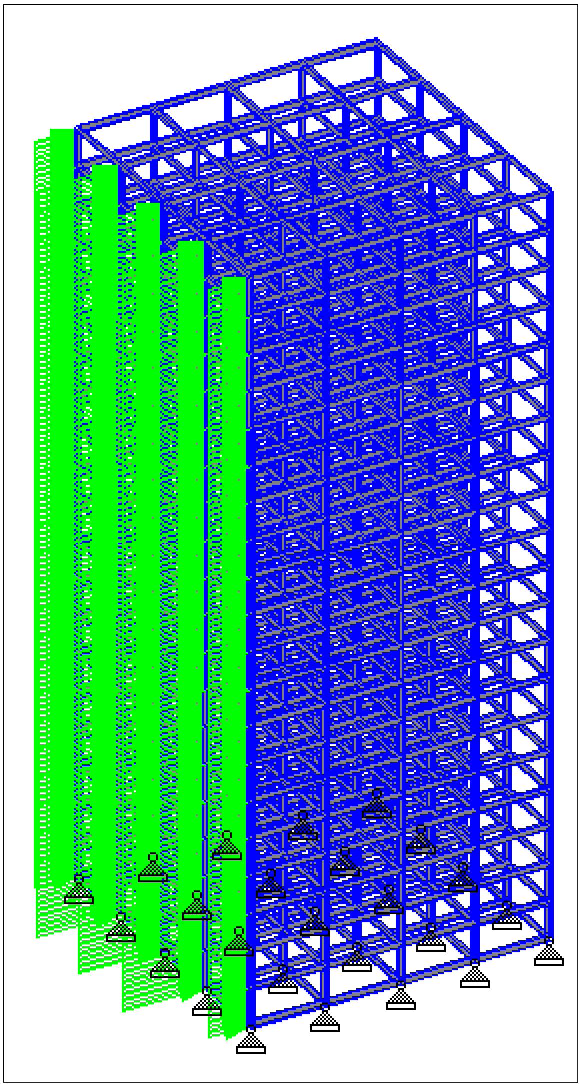

Figure 3 Wind Load Applied to 20 Storey Steel Building in StaadPro

3.3.3 Load Combination

The load combinations involving predominant wind load as per Eurocode 1:

1.35 Dead Load + 1.05 Live Load + 1.5 Wind Load

3.3.4 Material Property

The determination of material property of the multistory structure was done with optimization. The members were made as typical as possible, for ease of design and it is a usual practice for building design in terms of construction simplicity. These materials were checked against code BS 5950 with combination of Dead Load, Live Load and Wind Load in ultimate state as its benchmark for earthquake loading design. The details of material property can be shown in Table 4.

No of Storey	Building Height (m)	Beam Size	Column Size	Remarks
3	9	UB 457 x 191 x 82	UC 203 x 203 x 71	All Floors
5	15	UB 457 x 191 x 89	UC 254 x 254 x 107	1st - 3rd Floor
			UC 203 x 203 x 60	4th - 5th Floor
10	30	UB 457 x 191 x 89	UC 356 x 368 x 202	1st - 4th Floor
			UC 356 x 368 x 129	5th - 7th Floor
			UC 203 x 203 x 71	8th - 10th Floor
20	60	UB 533 x 210 x 109	UC 356 x 406 x 467	1st - 4th Floor
			UC 356 x 406 x 340	5th - 8th Floor
			UC 305 x 305 x 283	9th - 12th Floor
			UC 356 x 368 x 177	13th - 16th Floor
			UC 305 x 305 x 97	17th - 20th Floor
30	90	UB 610 x 229 x 113	UC 356 x 406 x 634	1st - 5th Floor
			UC 356 x 406 x 467	6th - 10th Floor
			UC 356 x 406 x 393	11th - 15th Floor
			UC 356 x 406 x 287	16th - 20th Floor
			UC 356 x 406 x 235	21st - 25th Floor
			UC 305 x 305 x 198	26th - 30th Floor

Table 5 Building Material Property

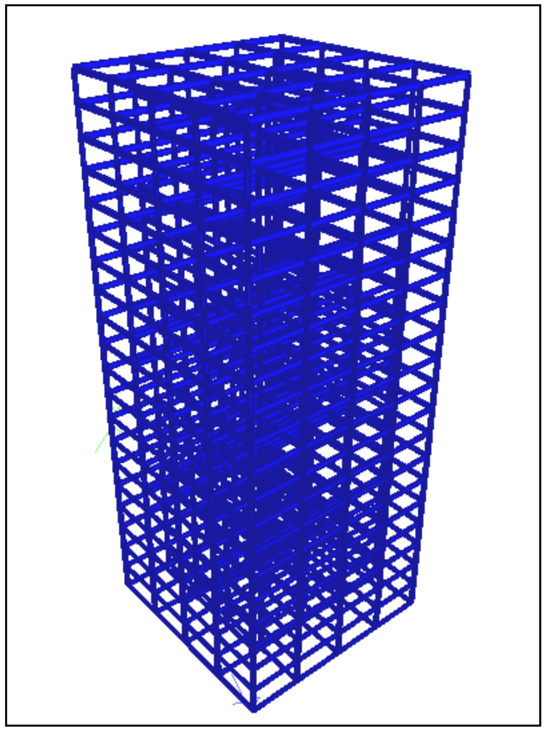

Figure 4 StaadPro Modelling of 20 Storey Building

3.4 SEISMIC ANALYSIS IN STAADPRO

3.4.1 Introduction

Unlike wind loading, earthquake loading also analyzed based on Eurocode 8: Design of structures for earthquake resistance. Where wind loading that is basically acting toward the column of the structure, earthquake loading is acting toward the mass of the floors in StaadPro. Seismic analysis will be including static analysis and dynamic analysis. Static analysis will be made by manual calculation while dynamic analysis will be calculated by using StaadPro.

3.4.2 Static Analysis

For calculation by using static analysis, base shear needed to be determined first based on response spectrum, which the function of structure natural period, and the total mass of the building. The response spectrum is based on the earlier section of methodology which is representing locally. The equation of base shear force and natural period are shown in the following equation.

Base Shear Force, $F_b = S_d(T_1) \cdot m \cdot \lambda$

where

S_d (T_1) is the ordinate of the design spectrum at period T_1;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered (s). T_1 may be approximated by the following expression:

$$T_1 = C_t \cdot H^{3/4}$$

where

Ct is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

m is the total mass of the building, above the foundation or above the top of a rigid basement (kN)

λ is the correction factor, the value of which is equal to: λ = 0,85

The calculation of storey mass and base shear force are shown in the following tables:

Storey	Brickwall (kN)	Beam (kN)	Column (kN)	Slab Load (kN)	Total Mass (kN)
3	0.0	193.1	52.2	3628.8	3874.1
2	3744.0	193.1	52.2	3628.8	7618.1
1	3744.0	193.1	26.1	3628.8	7592.0
				∑	19084.2

Table 6 3 Storey Building Mass Calculation

Storey	Brickwall (kN)	Beam (kN)	Column (kN)	Slab Load (kN)	Total Mass (kN)
5	0.0	209.5	22.1	3628.8	3860.4
4	3744.0	209.5	44.1	3628.8	7626.5
3	3744.0	209.5	61.4	3628.8	7643.8
2	3744.0	209.5	78.7	3628.8	7661.1
1	3744.0	209.5	78.7	3628.8	7661.1
				∑	34452.8

Table 7 5 Storey Building Mass Calculation

Storey	Brickwall (kN)	Beam (kN)	Column (kN)	Slab Load (kN)	Total Mass (kN)
10	0.0	209.5	26.1	3628.8	3864.5
9	3744.0	209.5	52.2	3628.8	7634.6
8	3744.0	209.5	52.2	3628.8	7634.6
7	3744.0	209.5	73.6	3628.8	7655.9
6	3744.0	209.5	94.9	3628.8	7677.3
5	3744.0	209.5	94.9	3628.8	7677.3
4	3744.0	209.5	121.8	3628.8	7704.1
3	3744.0	209.5	148.6	3628.8	7731.0
2	3744.0	209.5	148.6	3628.8	7731.0
1	3744.0	209.5	148.6	3628.8	7731.0
				∑	73041.0

Table 8 10 Storey Building Mass Calculation

Storey	Brickwall (kN)	Beam (kN)	Column (kN)	Slab Load (kN)	Total Mass (kN)
20	0.0	256.6	35.7	3628.8	3921.1
19	3744.0	256.6	71.4	3628.8	7700.8
18	3744.0	256.6	71.4	3628.8	7700.8
17	3744.0	256.6	71.4	3628.8	7700.8
16	3744.0	256.6	100.8	3628.8	7730.2
15	3744.0	256.6	130.2	3628.8	7759.7
14	3744.0	256.6	130.2	3628.8	7759.7
13	3744.0	256.6	130.2	3628.8	7759.7
12	3744.0	256.6	169.2	3628.8	7798.7
11	3744.0	256.6	208.2	3628.8	7837.6
10	3744.0	256.6	208.2	3628.8	7837.6
9	3744.0	256.6	208.2	3628.8	7837.6
8	3744.0	256.6	229.2	3628.8	7858.6
7	3744.0	256.6	250.2	3628.8	7879.6
6	3744.0	256.6	250.2	3628.8	7879.6
5	3744.0	256.6	250.2	3628.8	7879.6
4	3744.0	256.6	296.9	3628.8	7926.3
3	3744.0	256.6	343.6	3628.8	7973.0
2	3744.0	256.6	343.6	3628.8	7973.0
1	3744.0	256.6	343.6	3628.8	7973.0
				∑	152687.0

Table 9 20 Storey Building Mass Calculation

Storey	Brickwall (kN)	Beam (kN)	Column (kN)	Slab Load (kN)	Total Mass (kN)
30	0.0	266.0	72.8	3628.8	3967.7
29	3744.0	266.0	145.7	3628.8	7784.5
28	3744.0	266.0	145.7	3628.8	7784.5
27	3744.0	266.0	145.7	3628.8	7784.5
26	3744.0	266.0	145.7	3628.8	7784.5
25	3744.0	266.0	159.3	3628.8	7798.1
24	3744.0	266.0	172.9	3628.8	7811.7
23	3744.0	266.0	172.9	3628.8	7811.7
22	3744.0	266.0	172.9	3628.8	7811.7
21	3744.0	266.0	172.9	3628.8	7811.7
20	3744.0	266.0	192.0	3628.8	7830.9
19	3744.0	266.0	211.2	3628.8	7850.0
18	3744.0	266.0	211.2	3628.8	7850.0
17	3744.0	266.0	211.2	3628.8	7850.0
16	3744.0	266.0	211.2	3628.8	7850.0
15	3744.0	266.0	250.2	3628.8	7889.0
14	3744.0	266.0	289.1	3628.8	7928.0
13	3744.0	266.0	289.1	3628.8	7928.0
12	3744.0	266.0	289.1	3628.8	7928.0
11	3744.0	266.0	289.1	3628.8	7928.0
10	3744.0	266.0	316.4	3628.8	7955.2
9	3744.0	266.0	343.6	3628.8	7982.4
8	3744.0	266.0	343.6	3628.8	7982.4
7	3744.0	266.0	343.6	3628.8	7982.4
6	3744.0	266.0	343.6	3628.8	7982.4
5	3744.0	266.0	405.0	3628.8	8043.9
4	3744.0	266.0	466.5	3628.8	8105.3
3	3744.0	266.0	466.5	3628.8	8105.3
2	3744.0	266.0	466.5	3628.8	8105.3
1	3744.0	266.0	466.5	3628.8	8105.3
				∑	233332.9

Table 10 30 Storey Building Mass Calculation

No of Storey	Natural Period (s)	Response Spectrum (S_e/g)	Total Weight (kN)	Base Shear Force (kN)
3	0.44	0.5828	19084.18	709.04
5	0.65	0.5531	34452.81	1214.82
10	1.09	0.2961	73041.04	1378.75
20	1.83	0.1754	152687.05	1706.82
30	2.48	0.1273	233332.94	1892.99

Table 11 Building Base Shear Force Calculation

After getting the mass of each storey, the force applied to each storey can now be calculated. The force, F_i (in kN) at each storey are expressed as the following:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\Sigma z_j \cdot m_j}$$

where

z_i, z_j are the heights of the masses m_i m_j above the level of application of the seismic action (foundation or top of a rigid basement).

The details of the calculation of storey shear force are shown in the following tables:

Storey	Height, z_i (m)	Mass, m_i (kN)	$z_i \cdot m_i$	$z_i \cdot m_i / \sum z_j \cdot m_j$	Lateral Force (kN)	Uniform Lateral Force (kN/m)
3	9	3874.1	34866.9	0.3374	239	9.97
2	6	7618.1	45708.6	0.4423	314	13.07
1	3	7592.0	22775.9	0.2204	156	6.51
		∑	103351.4	1.0000	709	

Table 12 Storey Lateral Force Calculation For 3 Storey Building

Storey	Height, z_i (m)	Mass, m_i (kN)	$z_i \cdot m_i$	$z_i \cdot m_i / \sum z_j \cdot m_j$	Lateral Force (kN)	Uniform Lateral Force (kN/m)
5	15	3860.4	57906.2	0.2016	245	10.21
4	12	7626.5	91517.8	0.3187	387	16.13
3	9	7643.8	68794.0	0.2396	291	12.13
2	6	7661.1	45966.4	0.1601	194	8.10
1	3	7661.1	22983.2	0.0800	97	4.05
		∑	287167.6	1.0	1215	

Table 13 Storey Lateral Force Calculation For 5 Storey Building

Storey	Height, z_i (m)	Mass, m_i (kN)	$z_i \cdot m_i$	$z_i \cdot m_i / \sum z_j \cdot m_j$	Lateral Force (kN)	Uniform Lateral Force (kN/m)
10	30	3864.5	115933.8	0.1007	139	5.79
9	27	7634.6	206133.7	0.1791	247	10.29
8	24	7634.6	183229.9	0.1592	220	9.15
7	21	7655.9	160774.2	0.1397	193	8.03
6	18	7677.3	138190.6	0.1201	166	6.90
5	15	7677.3	115158.8	0.1000	138	5.75
4	12	7704.1	92449.3	0.0803	111	4.61
3	9	7731.0	69578.7	0.0604	83	3.47
2	6	7731.0	46385.8	0.0403	56	2.32
1	3	7731.0	23192.9	0.0201	28	1.16
		∑	1151027.6	1.0000	1379	

Table 14 Storey Lateral Force Calculation For 10 Storey Building

Storey	Height, z_i (m)	Mass, m_i (kN)	$z_i \cdot m_i$	$z_i \cdot m_i / \sum z_j \cdot m_j$	Lateral Force (kN)	Uniform Lateral Force (kN/m)
20	60	3921.1	235266.8	0.0504	86	3.58
19	57	7700.8	438945.4	0.0940	160	6.68
18	54	7700.8	415843.1	0.0890	152	6.33
17	51	7700.8	392740.7	0.0841	144	5.98
16	48	7730.2	371050.9	0.0794	136	5.65
15	45	7759.7	349184.6	0.0748	128	5.32
14	42	7759.7	325905.6	0.0698	119	4.96
13	39	7759.7	302626.6	0.0648	111	4.61
12	36	7798.7	280751.5	0.0601	103	4.28
11	33	7837.6	258642.3	0.0554	95	3.94
10	30	7837.6	235129.4	0.0503	86	3.58
9	27	7837.6	211616.5	0.0453	77	3.22
8	24	7858.6	188606.8	0.0404	69	2.87
7	21	7879.6	165471.3	0.0354	60	2.52
6	18	7879.6	141832.5	0.0304	52	2.16
5	15	7879.6	118193.8	0.0253	43	1.80
4	12	7926.3	95115.7	0.0204	35	1.45
3	9	7973.0	71757.2	0.0154	26	1.09
2	6	7973.0	47838.1	0.0102	17	0.73
1	3	7973.0	23919.1	0.0051	9	0.36
		∑	4670437.9	1.0000	1707	

Table 15 Storey Lateral Force Calculation For 20 Storey Building

Storey	Height, z_i (m)	Mass, m_i (kN)	$z_i \cdot m_i$	$z_i \cdot m_i / \sum z_j \cdot m_j$	Lateral Force (kN)	Uniform Lateral Force (kN/m)
30	90	3967.7	357091.8	0.0337	64	2.66
29	87	7784.5	677253.7	0.0639	121	5.04
28	84	7784.5	653900.2	0.0617	117	4.86
27	81	7784.5	630546.6	0.0595	113	4.69
26	78	7784.5	607193.0	0.0573	108	4.52
25	75	7798.1	584860.3	0.0552	104	4.35
24	72	7811.7	562445.9	0.0530	100	4.18
23	69	7811.7	539010.6	0.0508	96	4.01
22	66	7811.7	515575.4	0.0486	92	3.84
21	63	7811.7	492140.2	0.0464	88	3.66
20	60	7830.9	469852.7	0.0443	84	3.50
19	57	7850.0	447450.4	0.0422	80	3.33
18	54	7850.0	423900.4	0.0400	76	3.15
17	51	7850.0	400350.4	0.0378	71	2.98
16	48	7850.0	376800.4	0.0355	67	2.80
15	45	7889.0	355005.1	0.0335	63	2.64
14	42	7928.0	332975.9	0.0314	59	2.48
13	39	7928.0	309191.9	0.0292	55	2.30
12	36	7928.0	285407.9	0.0269	51	2.12
11	33	7928.0	261623.9	0.0247	47	1.95
10	30	7955.2	238656.6	0.0225	43	1.78
9	27	7982.4	215525.9	0.0203	38	1.60
8	24	7982.4	191578.6	0.0181	34	1.43
7	21	7982.4	167631.3	0.0158	30	1.25
6	18	7982.4	143684.0	0.0136	26	1.07
5	15	8043.9	120658.2	0.0114	22	0.90
4	12	8105.3	97263.8	0.0092	17	0.72
3	9	8105.3	72947.8	0.0069	13	0.54
2	6	8105.3	48631.9	0.0046	9	0.36
1	3	8105.3	24315.9	0.0023	4	0.18
		∑	10603470.5	1.0000	1893	

Table 16 Storey Lateral Force Calculation For 30 Storey Building

Below are the screenshot how the seismic loads are applied to the 20 storey steel buildings in StaadPro.

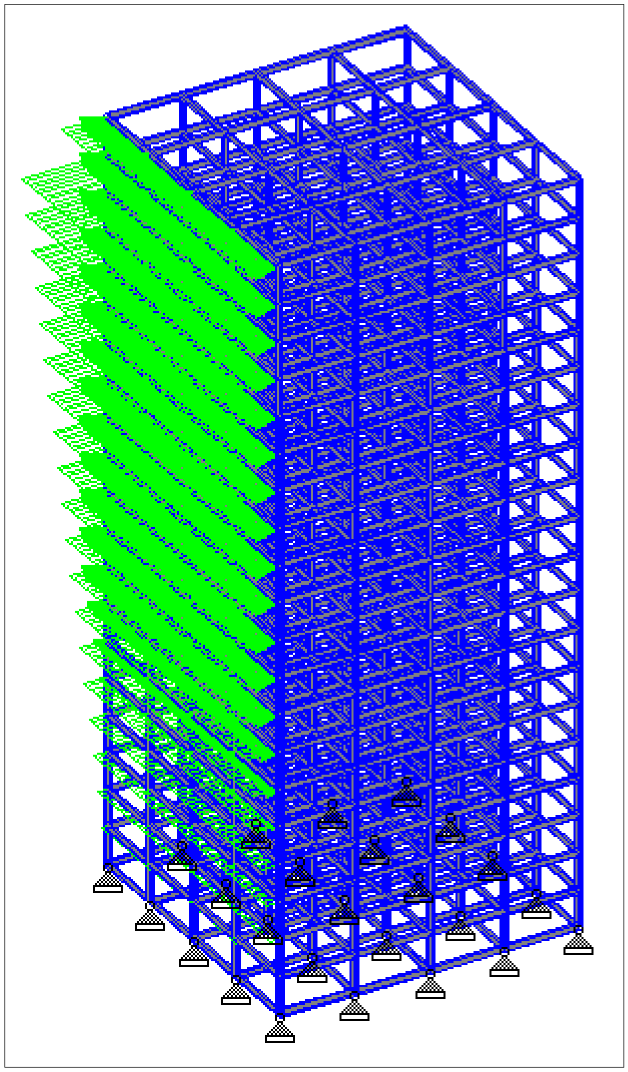

Figure 5 Seismic Loading Applied to 20 Storey Steel Building in StaadPro

3.4.3 Dynamic Analysis

For obtaining seismic load in terms of dynamic analysis, it is generated by using computer software. This is due to complex and tedious calculation when determining the values of eigenvalues, eigenvectors, modal participation factors and modal mass. There are different storey shear forces depending on its corresponding participation mode. The peak storey shear force in storey due to all modes considered is obtained by combining those due to each mode in accordance with modal combination; CQC (Complete Quadratic Combination) methods.

3.4.4 Load Combinations

When earthquake forces are considered on a structure, these shall be combined. In the elastic design of steel structures, the following load combinations shall be accounted for:

1) 1.0 (Dead Load) + 0.45 (Live Load) + 1.0(Earthquake Load)

3.5 MAPPING OUT RESEARCH TIMELINE

The framework and timeline for each activity involved in this research are presented in the following Gantt chart and key milestone;

No.	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14
1	Selection of Project Topic								Mid Semester Break							
2	Preliminary Research Work															
3	Submission of Extended Proposal Defence															
4	Proposal Defence															
5	Project Work Continues															
6	Submission of Interim Draft Report															
7	Submission of Interim Report															

Table 17 Gantt Chart FYP 1

No.	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14	15
1	Project Work Continues								Mid Semester Break								
2	Submission of Progress Report																
3	Project Work Continues																
4	Pre-SEDEX																
5	Submission of Draft Report																
6	Submission of Dissertation (Soft Bound)																
7	Submission of Technical Paper																
8	Oral Presentation																
9	Submission of Dissertation (Hard Bound)																

Table 18 Gantt Chart FYP 2

Week	Activities	Person Involved	Documentation Progress
End of Week 2	Selection of project title	FYP Coordinator	Project Selection
	Project Supervisor being assigned	FYP Supervisor	
	Confirmation of Project Title by Supervisor	Student	
End of Week 3	Read through journal and write literature review	Student	Literature Review
End of Week 4	Structuring the framework of study	Student	Methodology
End of Week 5	Extended Proposal draft preparation to be checked and amended by Project Supervisor	Student FYP Supervisor	Extended Proposal Defense
End of Week 6	Submission of Extended Proposal	Student	
End of Week 9	Project Defense	Student FYP Supervisor Internal Examiner	
End of Week 10	Development of spreadsheet template for structural analysis	Student	Methodology
End of Week 11	Validating spreadsheet template 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 Interirm Draft Report	Student FYP Supervisor	Interim Report
End of Week 14	Submission of Interim Report	Student FYP Supervisor	

Table 19 Key Milestone FYP 1

Week	Activities	Person Involved	Documentation Progress
End of Week 7	Analysis of Seismic Load (Static)	Student	Results
	Analysis of Seismic Load (Dynamic)	Student	
End of Week 8	Submission of Progress Report	FYP Stupervisor	Progress Report
End of Week 10	Interpret all data and results	Student	Discussion and Conclusion
End of Week 11	Pre-SEDEX	Student Examiner	Poster
End of Week 12	Submission of Draft Report	FYP Stupervisor	Dissertation Draft
End of Week 13	Submission of Dissertation (Softbound) and Techincal Paper	FYP Stupervisor	Project Dissertation
End of Week 14	Oral Presentation	Student FYP Supervisor External Examiner	VIVA
End of Week 15	Submission of Interim Report	Student FYP Supervisor	Project Dissertation

Table 20 Key Milestone FYP 2

3.6 TOOLS

- Microsoft Excel 2010
- Staad Pro 2007

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. The results are divided into two (w) sub-sections; the first one will be the analysis of wind loading, followed by earthquake loading of the same approach.

4.2 WIND LOADING ANALYSIS

Wind loads are treated as the benchmark in determining the building lateral strength. The maximum allowable lateral deflection according to Eurocode is H/200. The relationship between structural height and total deflection due to wind load are presented graphically in the following graph.

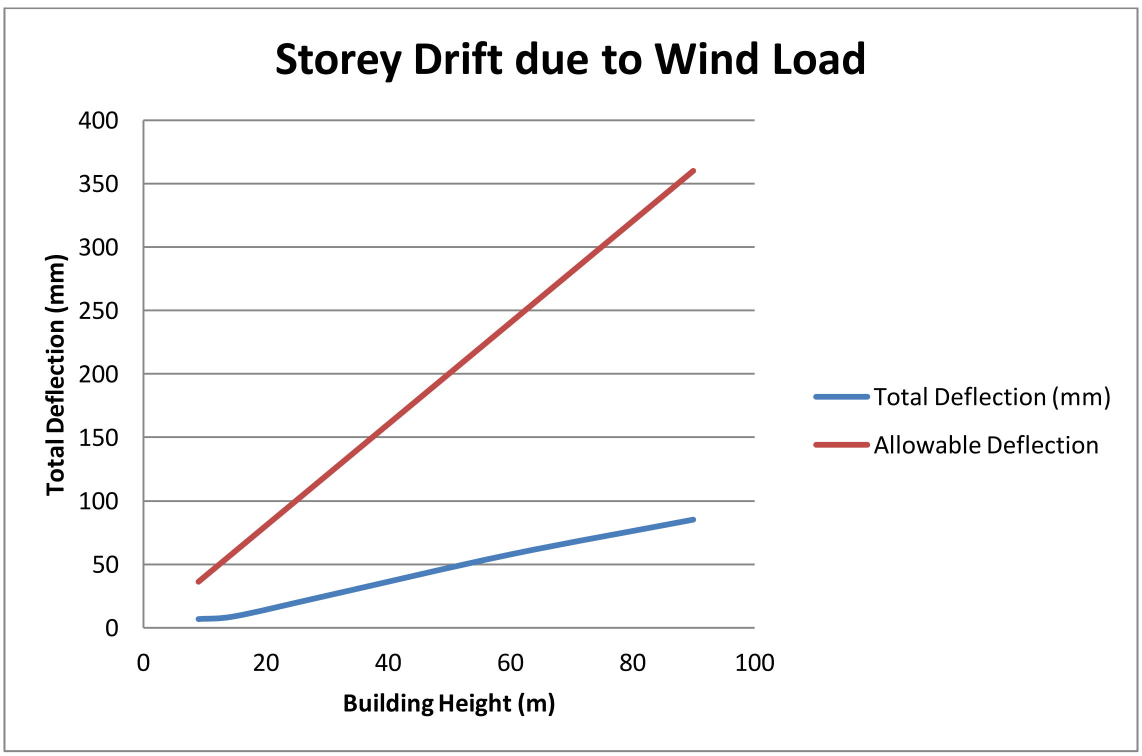

Figure 6 Building Drift due to Wind Load

The building deflection is almost directly proportional to the height of the building. This is due to uniform distribution among column of the buildings with respect to design optimization. The total deflection is in the allowable state with pass all the British Steel Code, which is required initially by the preliminary design.

4.3 SEISMIC LOADING ANALYSIS

Seismic Loading is analyzed and presented. The relationship between structural height and total deflection due to seismic load are presented graphically in the following table and graph.

No of Storey	Wind Load (mm)	Static Load (mm)	Dynamic Load (mm)	Allowable Deflection (mm)
3	6.89	21	20	45
5	9.14	39.6	38	75
10	25.22	52.4	48	150
20	57.59	68	60	300
30	85	75	65	450

Table 21 Building Drift due to Wind and Seismic Load

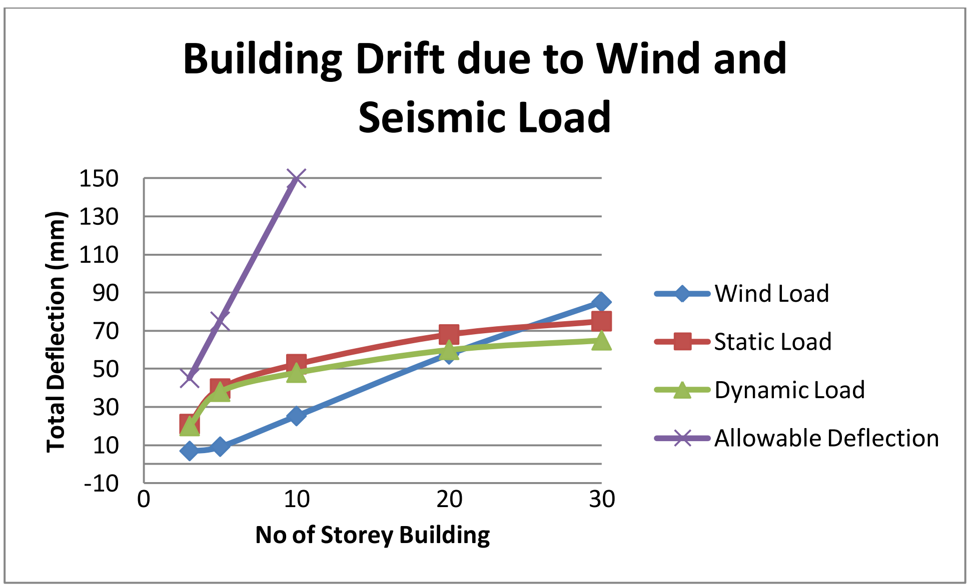

Figure 7 Building Drift due to Wind and Seismic Load

The deflection of the building due to seismic load are increasing with height due to more loads are applied. All the deflection did not exceed more than allowable deflection. However for 3 storey steel building, the deflection is nearing to the allowable deflection.

This is due to the plateau in the response spectrum where the building natural period is according to the highest spectral acceleration.

It's clear that the static analysis gives higher values for maximum displacement of the storey rather than dynamic analysis, especially in higher number of storey. As the number of storey goes up, deflection due to seismic loading is lower than wind loading. All the seismic loading is below the deflection limit of H/200. So it is safe to say that the building will not fail under seismic loading.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

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

1) Deflection due to wind loading is dependent on the ratio of exposed surface area to the number of columns
2) Deflection due to seismic loading is dependent on the total mass of each storey
3) However, these analyses are depending on building initial design. If the reserved strength is very high, the existing building might survive from seismic loading.
4) Static analysis is not sufficient for high rise building and it is necessary to provide dynamic analysis.
5) The difference of displacement values between static and dynamic analysis lower stories are insignificant but it increase in higher number of storey.
6) The results of equivalent static analysis are approximately uneconomical because values of displacement are higher than dynamic analysis.
7) As current condition in Malaysia, study shows that all structures are safe for seismic load in terms of deflection limit. These explain the zero documented structural failure so far due to these loads in Malaysia except some vibration on peninsular Malaysia due to far earthquake.

5.2 RECOMMENDATION

Analysis using static method is considered as very conventional and conservative in this decade. 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 economical. However, it is agreed that static analysis provide higher values of displacement compare to dynamic analysis. However, dynamic analysis is still considered as more practical method in analysing behaviour of structure towards lateral loads.

To extend the scope of this research, one may try to test it with other grades of steel and compare it with economical approach. In this way, we might get a better idea the difference of building performance based on the preliminary design with different steel yield strength.

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