High Rise Building: Vibration Control using Tuned Mass Damper

By

Lim Vicheaka

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfilment of the requirement for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)
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CERTIFICATE OF APPROVAL

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Approved by,

____
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December 2013

CERTIFICATE 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.

LIM VICHEAKA

Abstract

The availability of advance material, advance construction technology and powerful finite element analysis software have made modern tall building taller, slender and lighter which translate into dynamic force susceptibility. The main product of dynamic force is vibration in the building. The vibration causes discomfort to the occupants of the building and even damage the tall building to some extent. There are several method to mitigate this vibration problem. This project focus on using Tuned Mass Damper as the mitigation measure. The objective of this project is to apply the tuned mass damper characteristic onto a structural model of a building using finite element software in order to reduce its dynamic response and ultimately mitigate the vibration problem. The project cover the fundamental of structural dynamic, fundamental of tuned mass damper, dynamic analysis of a simple frame and a full frame structure and, finally, application of TMD on those structure. SAP2000 is used in this project as the main tool. A simple frame two storey 2D model known as Frame A and a full 3D model known as Frame B is used in this project for analysis. Preliminary dynamic analysis was done on Frame A using manual calculation and also using SAP2000. The result comparison between the two indicate that SAP2000 result is acceptable to use for analysis. Further time history analysis using periodic force with matching natural frequency applied on to the structure clearly shown the resonant effect. TMD was successfully design and integrated onto the structure in SAP2000 where the result show a reduction of structural dynamic response of Frame A and frame B.

Acknowledgements

My sincere thanks to my supervisor Dr. Teo Wee for helping and guiding me through this final year project smoothly and successfully. Mr. Ooi Shein Din, my supervisor at WEB STRUCTURES, for suggesting me this topic which helped me to learn a lot about structural dynamic. WEB STRUCTURES for allowing me to use their structural model to do analysis in this project.

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1 INTRODUCTION

1.1 Background of Study

When our ancestor first gained their skill to use tools, humanity began their quest to build higher. The Great Pyramid of Giza in Egypt was 146.5 meters tall when completed in 2560 BC and stood as the tallest man-made structure for over 3800 years without any help of heavy machineries. Standing at 55.86 meters tall, the Leaning Tower of Pisa in Italy is also a famous tall ancient structure completed in 1372. In the South East Asia region, Angkor Wat of Cambodia is standing at 65 meters tall which was completed in the 12th century mainly of sandstone. So from South America to the deepest jungle of South East Asia, we can find evidences from the past that indicate our desire to build higher.

The purpose of building higher used to be a religious one where people want to reach higher and nearer to their god. Nowadays tall building serve a lot more purposes. Firstly, it is accommodating the growth of population density in the cities around the world especially in area like Singapore and Hong Kong. Next is the maximization of profit and land use for the owner. Finally, tall building can serve as landmark. Malaysia built the PETRONAS Twin Tower which become the tallest building in the world for six year after its completion in 1998 and at the same time putting Malaysia as well as put Kuala Lumpur on the world map.

Looking at our present day, the desire to build higher does not show any sign of fading down. The growth gain its momentum in the 20th century after William LeBaron Jenny built a 10-story building using steel framework as the main structural support for the first time in history in 1885 (Taranath, 2012). This idea become the focusing point of civil industry where its application led to hundreds meter high tall building in the 20th century. Figure 1.1 shows the growth of building higher than 200 meters from the 1920s to early 21st century. It starts with only 2 buildings and continue to a significant development in the latter half of the 20th century. The number triple in the 2000s from 261 to 608. From then on, we can see continuous growth of the number of tall building around the world.

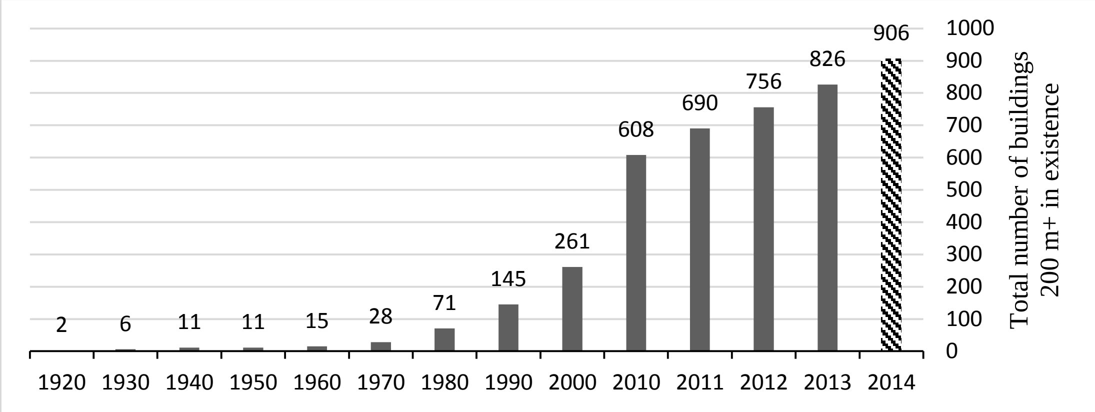


Figure 1.1 Total Number of tall buildings 200m+ in existence. (Brass, Wood, & Carver, 2013)

Concurrent to the breakthrough of structural system of tall building, other advancements also contributed to this rapid growth of tall building. Major findings in geotechnical engineering area such as advance soil modification technique play an important role by supporting the immense weight of growing tall building. Discovery made in material science provided the necessary element for tall building such as high strength steel and high strength concrete. Elevator technology also become more innovative to address the circulation issue in tall building. All of these and many other developments have made tall building become a more feasible and attractive option for developer. The evidence is clear as shown in figure 1.2. The average height of the 50 tallest building in existence keep on increasing year after year.

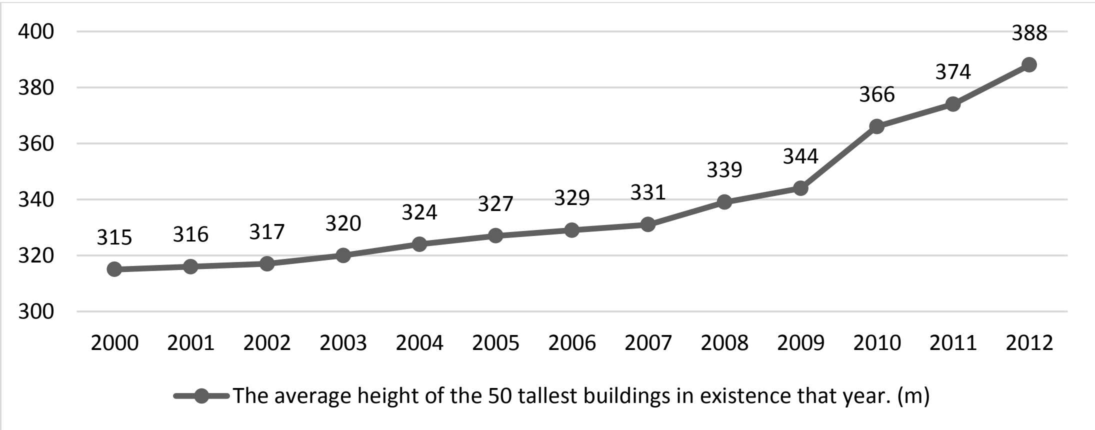


Figure 1.2 The average height of the 50 tallest buildings in existence from 2000 to 2012(Brass et al., 2013)

1.2 Problem Statement

These advancements for tall building do not come without its drawback. In the early 20th century, structural elements in tall building used to be very large because of the uncertainty in the design which, in turn, make the building stiffer as well as provide more damping because of more mass. With today's technology such as stronger materials and advance finite element structural analysis software, tall buildings are becoming taller and more slender but also less damped because of its small structural element and mass.

When tall building reach certain height depending on its location, dynamic factors start to affect the building. The two major forces that cause dynamic response of tall building are wind and seismic. The best known structural collapse due to wind was the Tacoma Narrows Bridge which occurred in 1940 at a wind speed of only about 19 m/s. It failed after it had developed a coupled torsional and flexural mode of oscillation. There are several different phenomena giving rise to dynamic response of structures in wind. These include buffeting, vortex shedding, galloping and flutter cause by eddies as shown in figure 1.3. Slender structures are likely to be sensitive to dynamic response in line with the wind direction as a consequence of turbulence buffeting. Transverse or cross-wind response is more likely to arise from vortex shedding or galloping but may also result from excitation by turbulence buffeting. Flutter is a coupled motion, often being a combination of bending and torsion, and can result in instability (Mendis, Ngo, Haritos, & Hira, 2007). Earthquake create ground movement that can shake the whole building which obviously cause dynamic response of tall building.

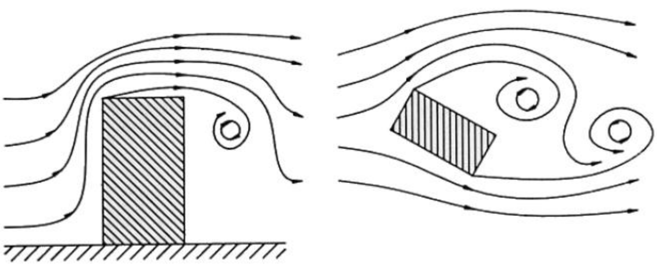


Figure 1.3 Generation of eddies, source of buffeting, vortex shedding, galloping and flutter (Mendis et al., 2007)

This dynamic response resulting in vibration of the building cause the occupants to feel discomfort because of the acceleration. There is no universally accepted standard for comfort criteria in tall building design. A considerable amount of research has however 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. These parameters include the occupant's expectancy and experience, their activity, body posture and orientation, visual and acoustic cues, and the amplitude, frequency, and accelerations for both the translational and rotational motions to which the occupant is subjected. Table 1.1 gives some guidelines on general human perception levels (Mendis et al., 2007).

LEVEL	ACCELERATION (m/sec^2)	EFFECT
1	<0.05	Humans cannot perceive motion
2	0.05-0.1	a) Sensitive people can perceive motion b) hanging objects may move slightly
3	0.1-0.25	a) Majority of people will perceive motion b) level of motion may affect desk work c) long term exposure may produce motion sickness
4	0.25-0.4	a) Desk work becomes difficult or almost impossible b) ambulation still possible
5	0.4-0.5	a) People strongly perceive motion b) difficult to walk naturally c) 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 1.1 Human Perception Levels on Tall building acceleration (Mendis et al., 2007)

There are many ways to mitigate this vibration problem such as stiffening the structure, increasing the mass, change the aerodynamic of the structure and auxiliary damping device. Stiffening the structure is comparable to turning back to early 20th century design method where the structure will have huge structural element which is not economically attractive in modern society. Increasing the mass also mean wasting floor spacing which translate into money. While changing the aerodynamic of the structure basically mean the building architectural design has to change. These options are not suitable which is why we should venture into auxiliary damping system. Types of damping system that can be implemented include, passive, active, semi-active and hybrid systems. Some example of passive systems are Tuned Mass

Damper (TMD), Tuned Liquid Damper (TLD), Friction Device, Metallic Yield Devices, Viscous Elastic Damper and Viscous Fluid Damper. Examples of active control system are Active Mass Damper (AMD), Active Tendon System, Active Bracing System with hydraulic actuator and Pulse Generation System. Examples of semi-active system are Semi-active TMD, Semi-active TLD, Semi-active Friction damper, Semi-active Vibration Absorber, Semi-active Stiffness Control Device, Electro rheological Damper and Magneto Rheological dampers (Cheng, 2008).
Tuned Mass Damper or TMD is the chosen mitigation method in this paper. TMD is a device consisting of a mass, a spring, and a damper that is attached to a structure in order to reduce the dynamic response of the structure. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will resonate out of phase with the structural motion. Energy is dissipated by the damper inertia force acting on the structure (Connor, 2003). As a result, it reduce the vibration of the building.

The advantages of this system are external power is not needed[1], provide large damping force and can be install on existing structure. Some draw backs also exist such as its limitation to a narrow frequency, sensitive to mistuning and need a dedicated area to house the system.

With its constant economic growth, Malaysia is inevitable from vertical expansion where we will see more tall building being erected. The recent increment of seismic activities in the region has made Malaysia a perfect example where TMD may prove to be useful in the future. The landscape of building design regulation will shift toward a safer design where seismic design will be enforced. Understanding TMD will pave the way to other systems such as AMD or HMD where it will be both applicable to the existing building as well as new building.

[1] The system is activated by the motion of the structure.

1.3 Objective of Study

To be able to apply the tuned mass damper characteristic onto a complete structural model of a building using finite element software in order to reduce its dynamic response and ultimately mitigate the vibration problem.

1.4 Scope of Study

This study will cover:

- Fundamental of structural dynamic
- Dynamic Response Analysis of a Simple Frame structure
- Dynamic Response Analysis of a Full Frame structure
- Fundamental of Tuned Mass Damper (TMD)
- Application of TMD on simple frame structure and full frame structure
- Effectiveness of TMD on the analysis model

2 LITERATURE REVIEW AND THEORY

2.1 Fundamental of Vibration

Vibration is the periodic motion of a body or system of connected bodies displaced from a position of equilibrium. The simplest type of vibrating motion is undamped free vibration, represented by the model shown in figure 2.1. The block has a mass *m* and is attached to a spring having a stiffness k. Vibration occurs when the block is released from a displaced position *x* so that the spring pulls on the block. The block will attain a velocity such that it will proceed to move out of equilibrium when $x = 0$ and provided the supporting surface has no friction, oscillation will continue indefinitely (Hibbeler, 2006).

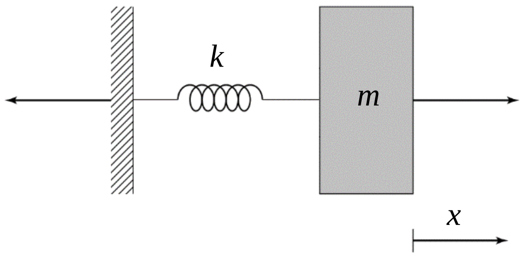


Figure 2.1 Undamped Spring Mass System

Equilibrium equation

$$-kx = m\ddot{x}$$

The standard form give

$$\ddot{x} + \omega_n^2 x = 0 \tag{2-1}$$

Where $\omega_n = \sqrt{\frac{k}{m}}$ is called the natural frequency expressed in rad/s.

Equation 2-1 is a homogeneous, second-order, linear, differential equation with constant coefficient.

So the general solution will be $x = A\sin\omega_n t + B\cos\omega_n t$ (2-2)

Where it can be express as $x = C\sin(\omega_n t + \phi)$ (2-3)

If this equation is plotted on an x-versus-$\omega_n t$ axis, the graph shown in figure 2.2 is obtained

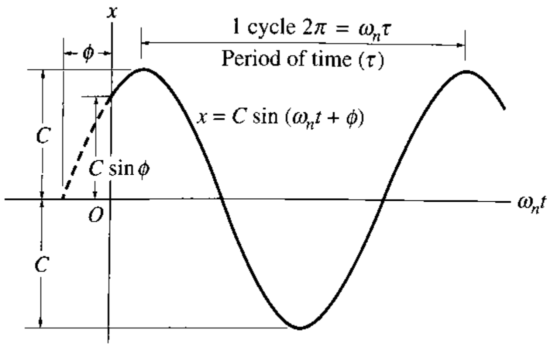


Figure 2.2 Periodic Wave

So vibration can be translated in wave form. The dynamic response of tall buildings are similar to this manner. The purpose of this study is to reduce the wave magnitude and bring the structure to equilibrium or to an acceptable acceleration.

For this particular case, damping is need to reduce the wave motion. The vibration considered before has not included the effects of damping in the system, and as a result, the solutions obtained are only in close agreement with the actual motion. Since all vibrations die out in time, the presence of damping forces should be included in the analysis as shown in figure 2.3

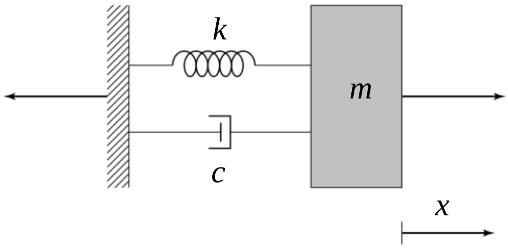


Figure 2.3 Damped Spring Mass System

Equilibrium equation

$$m\ddot{x} + c\dot{x} + kx = 0 \qquad (2\text{-}4)$$

Equation 2-4 is a homogeneous, second-order, linear, differential equation

For this study, only the underdamped system result is discussed

$$x = D\left[e^{-(c/2m)t}\sin(\omega_d t + \phi)\right] \tag{2-5}$$

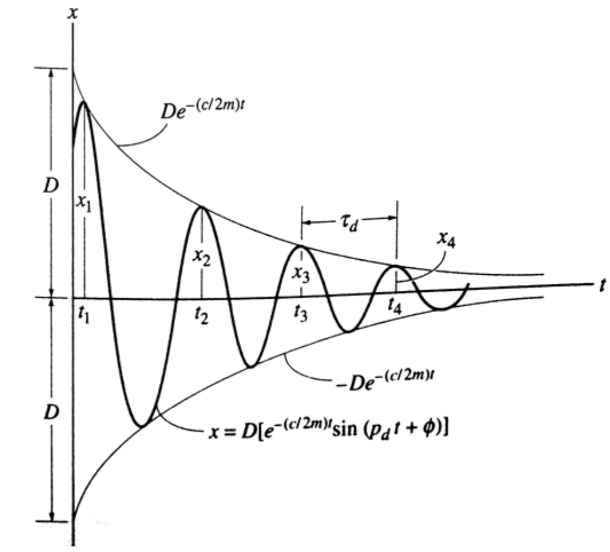


Figure 2.4

Figure 2.4 show the effect of damping *c* on the wave of the vibration where it can be significantly reduced. The initial limit of motion, *D*, diminishes with each cycle of vibration, since motion is confined within the bounds of the exponential curve $De^{-(c/2m)t}$ and $-De^{-(c/2m)t}$.

Tall buildings naturally have its damping but the value is getting smaller which the vibration exceed the comfort level for the occupant. Small mass with frequency tuned to a particular structural frequency to resonate out of phase with the structural motion create the idea for TMD.

It can be explain by using a Single Degree of Freedom (SDOF) system attached to TMD.

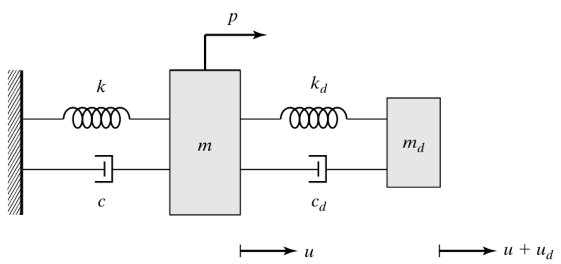


Figure 2.5 SDOF-TMD system

The governing equation of motion are given by

Primary mass $(1+\bar{m})\ddot{u}+2\xi\omega\dot{u}+\omega^2 u=\frac{p}{m}-\bar{m}\ddot{u}_d$ (2-6)

Tuned mass $\ddot{u}_d+2\xi_d\omega_d\dot{u}_d+\omega_d^2 u_d=-\ddot{u}$ (2-7)

The purpose of adding the mass damper is to limit the motion of the structure when it is subjected to a particular excitation. The design of the mass damper involves specifying the mass m_d, stiffness k_d, and damping coefficient c_d (Connor, 2003).

The spring-mass model shown a horizontal system which does not accurately represent a building. A simple multi degree of freedom frame structure can be convert into a dynamic model using a lump mass system. The mass of the N story frame is lumped at the floor levels with m_j denoting the mass at the j^{th} floor. This system has N degree of freedom: $u_1, u_2, \dots, u_N$ as shown in figure 2.6. These lumped mass represent the mass in spring mass system.

The stiffness of the spring is represented by the story stiffness which is the sum of the lateral stiffnesses of all columns in the story (Chopra, 1997). For a story of height h and a column with modulus E and a second moment of area I_c, the lateral stiffness of a column with fixed ends, implied by the shear-building idealization is $12EI_c/h^3$. Thus the story stiffness is

$k_j=\sum_{columns}\frac{12EI_c}{h^3}$ (2-8)

2.2 Type of TMD

On the global market there are a number of manufacturers that are specialize in vibration control equipment that can manufacture tune mass damper such as TVS of the UK, Vibratec of Sweden and Maurer Sohne of Germany. There are 3 main types of TMD ("MAURER Tuned Mass and Viscous Dampers," 2011) available:

- Vertical Acting TMD as shown in figure 2.6 are used for controlling vertical vibration
- Horizontal Acting TMD as shown in figure 2.7 are used for controlling horizontal vibration
- Pendulum TMD as shown in figure 2.8 are used for controlling horizontal vibration

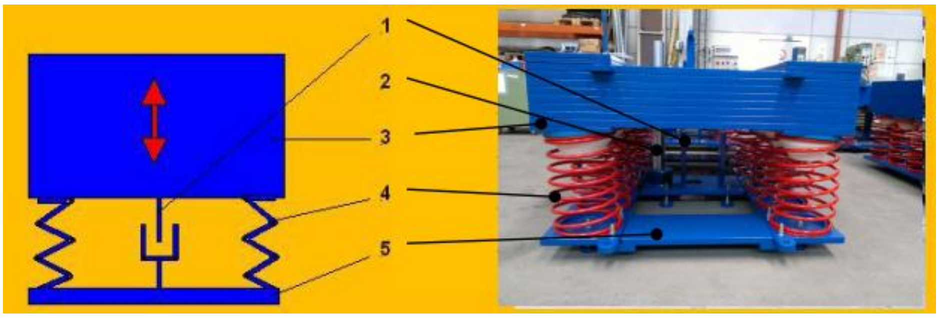


Figure 2.6 Vertical Acting TMD

Different types of TMD exist to suite a wide range of situation. Beside the standard products, TMD can also be custom made to control both horizontal and vertical vibration if needed.

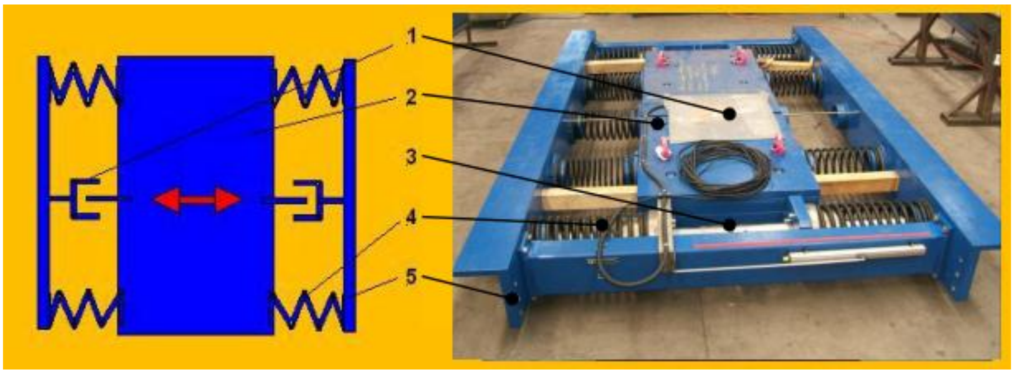


Figure 2.7 Horizontal Acting TMD

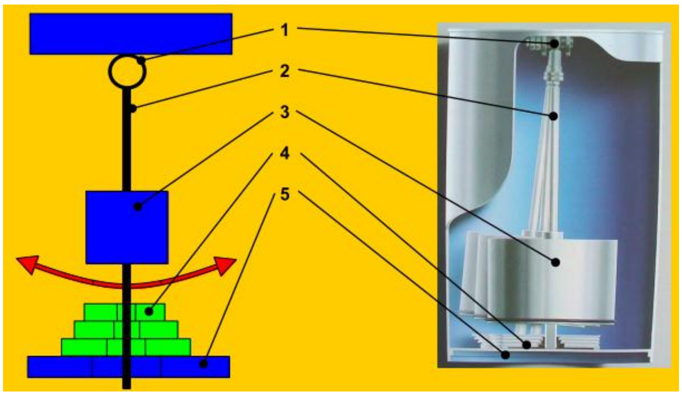


Figure 2.8 Pendulum TMD

2.3 Adoption and effectiveness of TMD

The tallest residential tower in Iran with 56 stories reaching up to 170 meters was used to study the effect of tuned mass damper. The study was conducted with the help of SAP2000 model. Three real earthquake with different magnitude, epicentre distance and duration was taken into consideration. The second and third modes have the main role in the structural translation so the TMD used is tuned to these modes

(Okhovat, Rahimian, & Ghorbani-tanha, 2006). The result of the study showed that even a relatively small mass of about 90 tons TMD compared to the building total mass of 400000 tons can reduce the displacement and acceleration response for about 25%.

Recent study by (Sanhyun, Chung, Kim, & Woo, 2012) indicates the rise of damping device usage in South Korea and its intention to enter the world vibration control device market. Table 2.1 shows super-tall building in South Korea that use vibration control technology.

Applied Construction	Device Type	The Year of Installation
Incheon Int'l Airport Control Tower	HMD	1999
Yangyang Int'l Airport Control Tower	TMD	2000
Galleria Palace	VED	2003
Centum City	TMD	2004
Hyundai Hyperion	TLD	2005
Lotte Hotel	AMD	2007
Posco The First World	TLCD	2008
Posco Construction HQ Office Building	TMD	2009

Table 2.1 Super-Tall Building Vibration Control Technology Application Status in Korea

A test was done in South Korea on a building call TechnoMart21 which suffer wind-induced vibration generating acceleration as high as 7cm/s^2. After installing TMD, another acceleration test was conducted showing the highest of only 3 cm/s^2 lower than half of the acceleration before installing TMD.

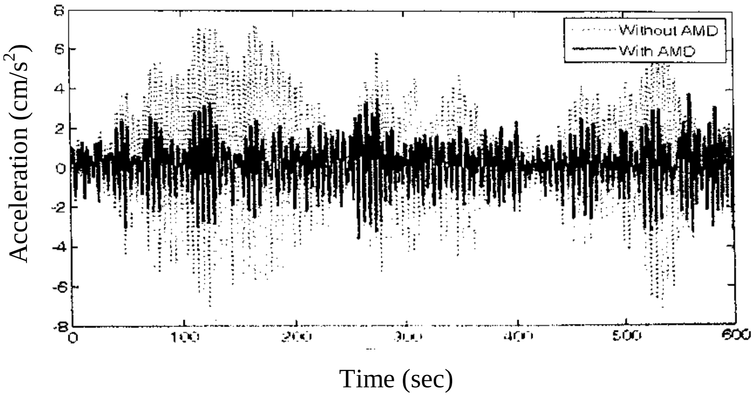


Figure 2.9 TechnoMart21 Acceleration Test Result

Probably the most famous TMD is the one in Taipei 101 which it serve as an architectural element of the tower. Similar to the tower in Tehran, this analysis was also done with the help of SAP2000 through time history analysis. The world biggest Tuned Mass Damper[2], outriggers, supercolumns, high-strength concrete and steel, moment resisting frame and well optimized aerodynamic shape are the key structural elements that made Taipei 101 a reality, especially for the region of Taiwan, which is very susceptible to catastrophic typhoons and earthquakes (Kourakis, 2007). This building is 508 meters high. The analysis was done using an equivalent 10 degrees of freedom model. The first mode of the model was calibrated to match the known period of 6.8sec.

Without the TMD, maximum acceleration of the model is 7.7 cm/s^2 which far larger than the acceptable acceleration of 5 cm/s^2.

With the TMD, maximum acceleration reduced to 4.935 cm/s^2 translate to about 35% performance increase in term of dynamic response.

2.4 Den Hartog's Optimization Criteria

TMD efficiency in reducing structural response can be gained by following the basic development of Den Hartog for the simple case where the structural system is considered undamped (C=0) and is subject to a sinusoidal excitation with frequency ω ($f(t) = P_0 \sin \omega t$)(Soong & Dargush, 1997). This procedure compare the dynamic effect of a TMD with the static deflection produced by the maximum force applied statically to the structure. The dynamic amplification factor for an undamped structural system,R, is

$$R = \frac{y_{max}}{y_{st}} = \sqrt{\frac{(\alpha^2 - \beta^2)^2 + (2\zeta_a \alpha\beta)^2}{[(\alpha^2 - \beta^2)(1 - \beta^2) - \alpha^2\beta^2\mu]^2 + (2\zeta_a \alpha\beta)^2(1 - \beta^2 - \beta^2\mu)^2}}$$

Where $\beta = \omega_f / \omega$ External force excitation frequency ratio

$\alpha = \omega_d / \omega$ TMD frequency ratio

$\mu = m_d / m$ TMD mass ratio

$\omega_d^2 = k_d / m_d$ Squared natural frequency of TMD

[2] 730 tons

$\omega^2 = k/m$ Squared natural frequency of structural system

$\zeta_d = \frac{c}{c_c} = c/2m\omega_d$ Damping ratio of TMD

Figure 2.10 show a plot of R as a function of the frequency ratio β for $\alpha = 1$ (tuned case), $\mu = 0.05$, and for various values of TMD damping ratio ζ_d.

Without structural damping, the response amplitude is infinite at two resonant frequencies of the combined structure/TMD system. When the TMD damping becomes infinite, the two masses are virtually fused to each other and the result is a SDOF system with mass $1.05m$ so that the amplitude at resonant frequency become infinite again. Therefore, somewhere between these extremes there must be a value of ζ_d for which the peak becomes a minimum.

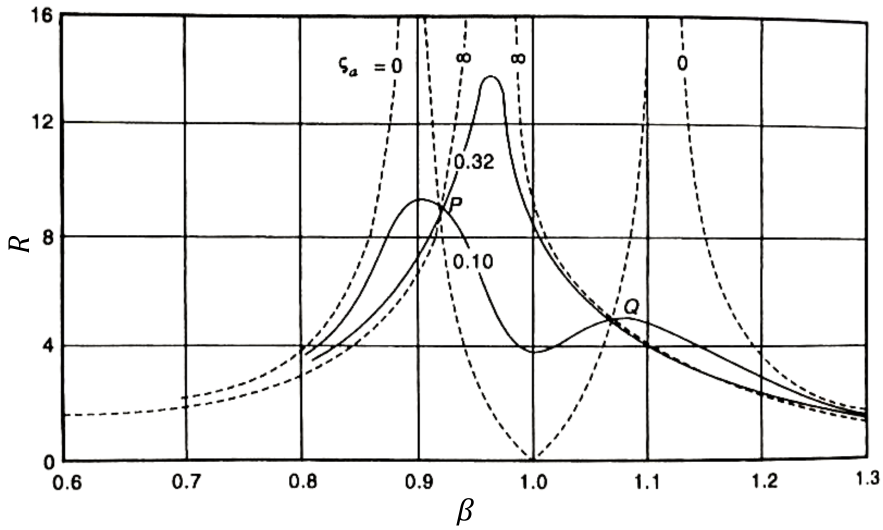


Figure 2.10 Amplification Factor as function of β (Soong & Dargush, 1997)

There are two points (P and Q) on Figure 2.10 at which R is independent of damping ratio ζ_d and the minimum peak amplitude can be obtained by first properly choosing α to adjust these fixed points to reach equal heights. The optimum frequency ratio α following this procedure is determined as

$$\alpha_{opt} = \frac{1}{1+\mu}$$

Which gives the amplitude at P or Q

$$R = \sqrt{1 + \frac{2}{\mu}}$$

A good estimate for ζ_{opt} can be determined as the average of two values which make the fixed points P and Q maxima on figure 2.10 giving

$$\zeta_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}}$$

The maximum amplification factor and optimum absorber parameters are summarized in Table 2.2 for a variety of excitations and response quantities density is assumed.

Case	Excitation		Optimized absorber parameter	
	Type	Applied to	α_{opt}	ζ_{opt}
1	Periodic Force	Structure	$\frac{1}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)}}$
2	Acceleration	Base	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)(1-\mu/2)}}$
3	Random Force	Structure	$\frac{\sqrt{1+\mu/2}}{1+\mu}$	$\sqrt{\frac{\mu(1+3\mu/4)}{4(1+\mu)(1+\mu/2)}}$
4	Random Acceleration	Base	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\sqrt{\frac{\mu(1-\mu/4)}{4(1+\mu)(1-\mu/2)}}$

Table 2.2 Optimum Absorber Parameters attached to undamped SDOF Structure (Warburton, 1982)

2.5 Mass of TMD

The mass ratio μ of the TMD mass to the kinetic equivalent structural mass has to be sufficient. For small ratios ($\mu \leq 0.025$) big vibration amplitudes of the TMS mass relatively to the structure are resulting. This can create a space problem for proper integration of the TMD in the available structural gap, but also the TMD gets usually much more expensive due to more and bigger springs.

In addition, a small mass ratio is decreasing the effective range of the TMD. The TMD mass movements are significantly smaller for bigger ratios ($\mu \geq 0.025$) and the effective range for a 100% TMD efficiency around the resonance frequency is greater.

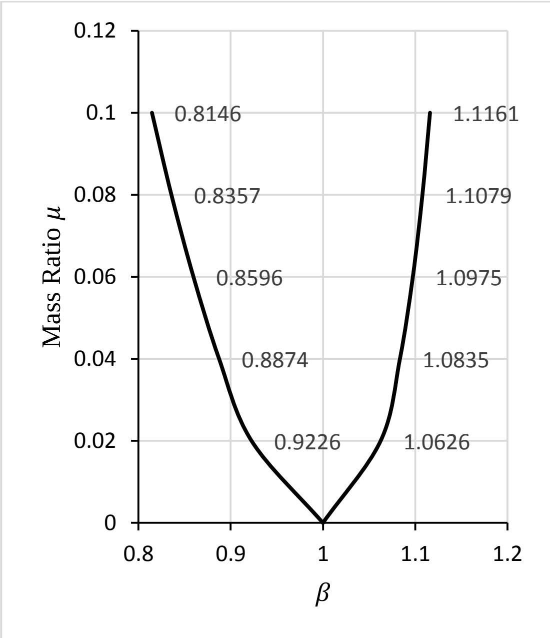


Figure 2.11 Frequency range with respect to μ

3 METHODOLOGY

3.1 Finite Element Analysis Software (FEA)

This study is done using a structural finite element analysis software called SAP2000 which are capable of conducting dynamic analysis.

3.2 Model Definition

Two analysis models will be used in this study. First, Frame A, is a simple x-z plane frame two stories model which will be used for manual dynamic response calculation and for preliminary TMD application study using FEA software as shown in figure 3.1. Second, Frame B, is a full 3D frame analysis model will be used for the final study of the TMD as shown in figure 3.1.

3.2.1 Frame A

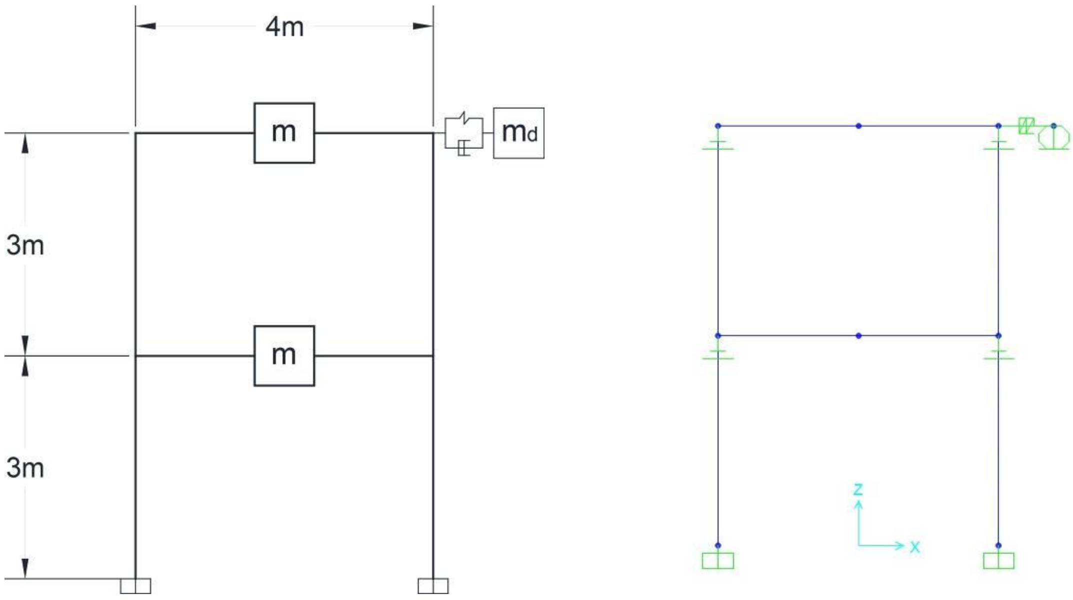


Figure 3.1Frame A (Right: Frame A in SAP2000)

All elements of frame A are UC203x203x71 standing 6 metre high and 4 metre wide. m is 50 tons each. Tuned mass damper is attached to the structure through spring and dashpot link at the top floor.

3.2.2 Frame B

A twenty seven storey composite steel frame building with specific dimensions as shown in table 3.1. Figure 3.2 shows the typical structural plan of the repeated floors for the total 27 storey.

<table>
<tr><th rowspan="3">Floor</th><th colspan="6">Dimension</th></tr>
<tr><th colspan="2">Columns</th><th colspan="4">Hollow Steel Beams</th></tr>
<tr><th>C1*</th><th>C2**</th><th>B1</th><th>B2</th><th>B3</th><th>B4</th></tr>
<tr><td>1-6</td><td>500x2300x20</td><td rowspan="5">400x900x80</td><td rowspan="5">500x625x20</td><td rowspan="5">750x650x40</td><td rowspan="5">300x900x80</td><td rowspan="5">500x800x80</td></tr>
<tr><td>7-12</td><td>500x1850x20</td></tr>
<tr><td>13-17</td><td>500x1400x20</td></tr>
<tr><td>18-23</td><td>500x950x20</td></tr>
<tr><td>24-27</td><td>500x500x20</td></tr>
</table>

Table 3.1 Dimension of steel frame elements of the building (mm)
**Composite Column*
***Hollow Steel Column*

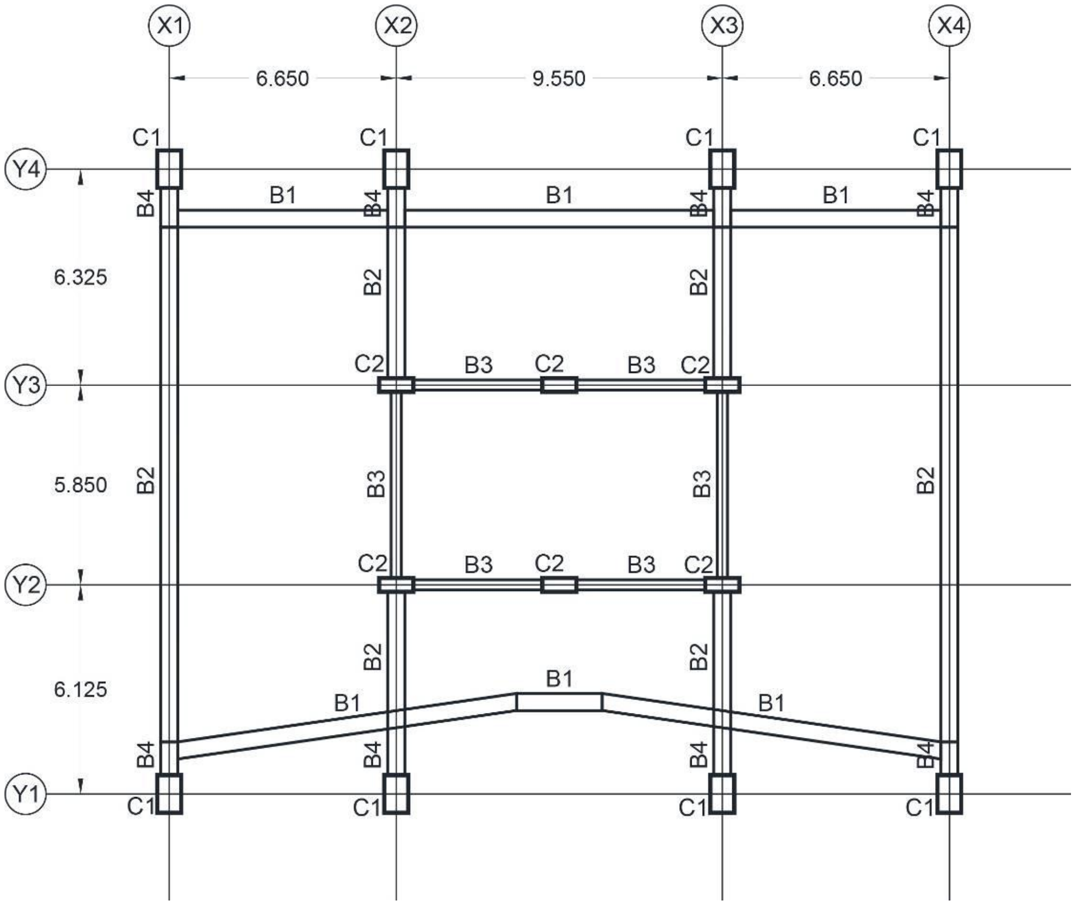


Figure 3.2 Typical structural plan of each floor of the 27 storey building model

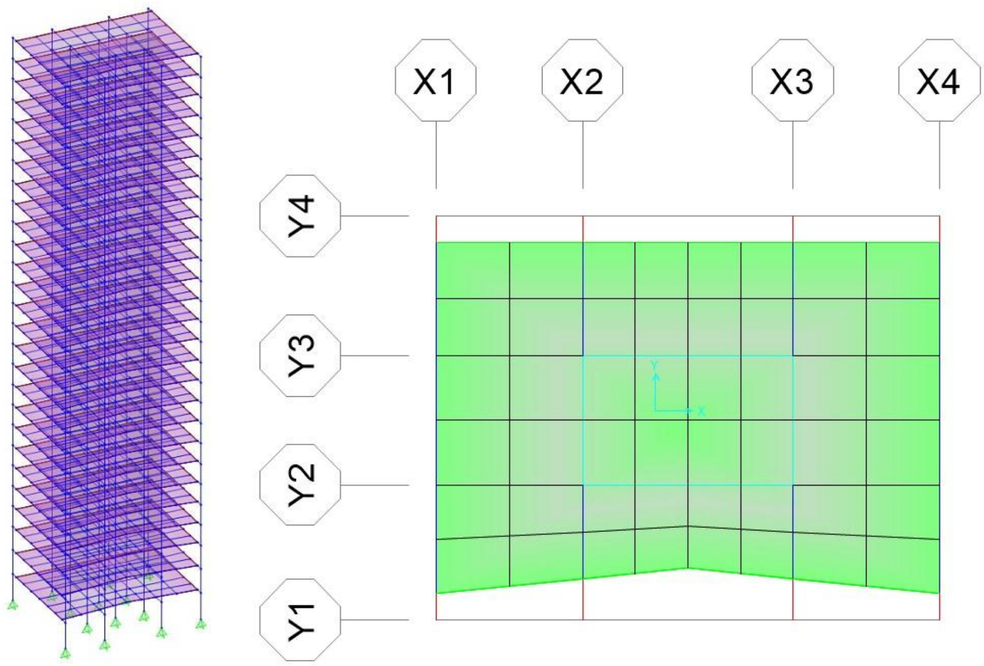


Figure 3.3 Frame B created in SAP2000

Adjusted periodic load is applied to the top floor along X-axis direction to simulate the resonant effect caused by wind or seismic load. The TMD is attached to the structure on the top floor acting along X-axis direction as shown in figure 3.4.

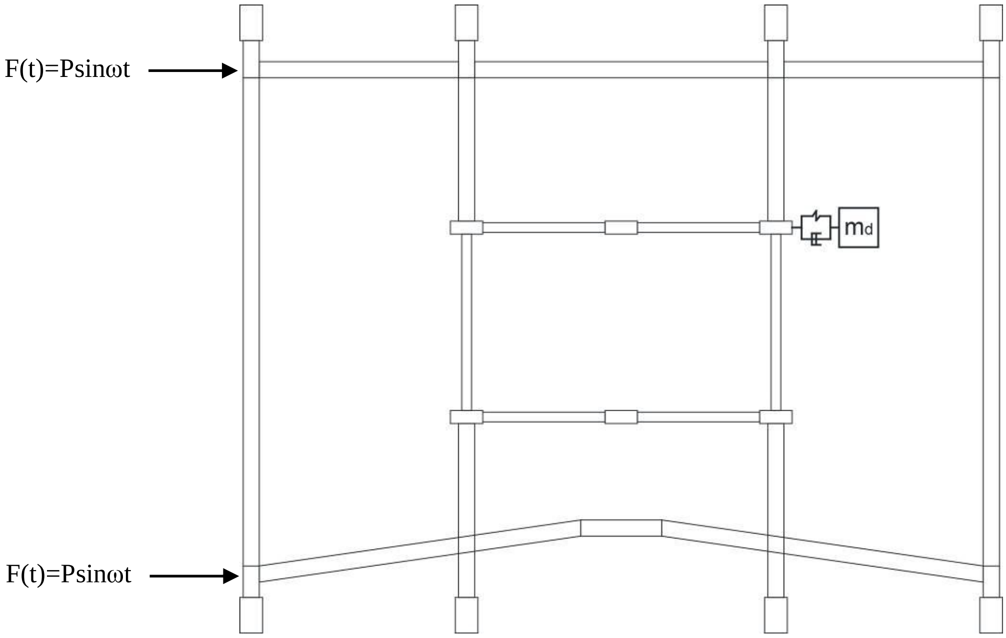


Figure 3.4 Frame B top floor force direction and TMD location

3.3 Dynamic response of frame (A) using manual calculation

A simple frame model can be represented by spring mass model as shown below.

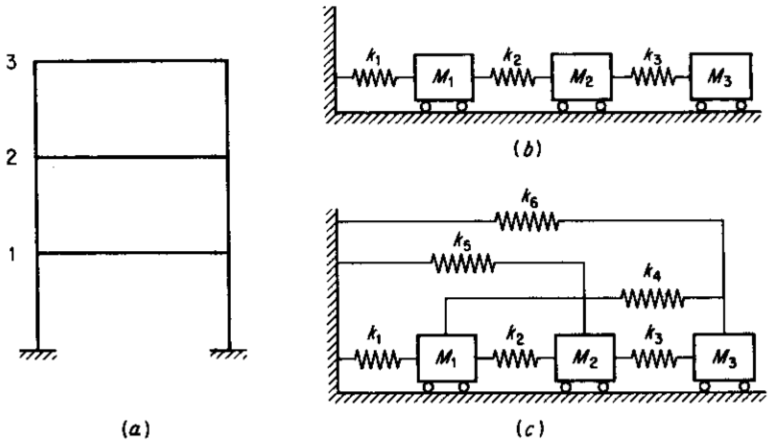


Figure 3.5 Simple frame model to spring mass model (Biggs, 1964)

Depending on the characteristic of the structure, the arrangement of the spring mass model can be different. For example, if the girder rigidity approaches infinity, the system (considering only horizontal motion) may be represented as show in figure 3.3b. On the other hand, if the girders are flexible, a proper representation is as shown in figure 3.3c.

Assuming girders rigidity of frame (A) are approaching infinity, it can be represented as below.

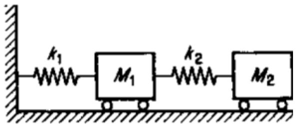


Figure 3.6 Frame (A) spring mass model

The equations of motion for this system are

$$M_1\ddot{y}_1 + k_1 y_1 - k_2(y_2 - y_1) = 0$$
$$M_2\ddot{y}_2 + k_2(y_2 - y_1) = 0 \quad (3\text{-}1)$$

If the system is vibrating in a normal mode (natural mode), the two displacements are harmonic and in phase, and may be expressed by:

$$y_1 = a_1 \sin\omega(t+\alpha) \qquad \ddot{y}_1 = -a_1\omega^2 \sin\omega(t+\alpha)$$
$$y_2 = a_2 \sin\omega(t+\alpha) \qquad \ddot{y}_2 = -a_2\omega^2 \sin\omega(t+\alpha)$$

Substitute it in equation (3-1) to obtain

$$-M_1 a_1\omega^2 + k_1 a_1 - k_2(a_2 - a_1) = 0$$
$$-M_2 a_2\omega^2 + k_2(a_2 - a_1) = 0$$

Or

$$(-M_1\omega^2 + k_1 + k_2)a_1 - k_2 a_2 = 0$$
$$-k_2 a_1 + (-M_2\omega^2 + k_2)a_2 = 0 \quad (3\text{-}2)$$

In order for the amplitudes to have any values other than zero (n necessary condition for a natural mode), the determinant of the coefficients must be equal to zero.

$$\begin{vmatrix} -M_1\omega^2 + k_1 + k_2 & -k_2 \\ -k_2 & (-M_2\omega^2 + k_2) \end{vmatrix} = 0$$

Expanding this determinant gives the equation

$$(-M_1\omega^2 + k_1 + k_2)(-M_2\omega^2 + k_2) - (k_2)^2 = 0$$

or

$$(\omega^2)^2 - (\frac{k_2}{M_2} + \frac{k_1 + k_2}{M_1})\omega^2 + \frac{k_1 k_2}{M_1 M_2} = 0$$

Frame (A) consist of $M_1 = M_2 = M$ and $k_1 = k_2 = k$

$$(\omega^2)^2 - (\frac{3k}{M})\omega^2 + \frac{k^2}{M^2} = 0$$

The two root of this equation are

$$\omega = 0.618\sqrt{\frac{k}{M}}$$

$$\omega = 1.618\sqrt{\frac{k}{M}}$$

Where the natural frequencies of the two normal modes is $f = \omega/2\pi$ and $T = \frac{1}{f}$

3.4 TMD Design procedure for Frame A and Frame B

The design of TMD will be based on Den Hartog optimization criteria assuming that the structure fit the condition for the optimization criteria. Mass ratio for the TMD will be 0.04 for this study. Depending on the available space, larger mass can be use where it will increase the efficiency range. Then optimum frequency ratio $\alpha_{opt} = \frac{1}{1+\mu}$ can be calculated. Frequency ratio give up the frequency of the TMD where it can be used to calculate spring constant k of the system. The damping ratio of the TMD will be calculated by using the mass ratio $\zeta_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}}$. Damping coefficient can be found by using the damping ratio $c = 2\zeta_{opt} m\omega$.

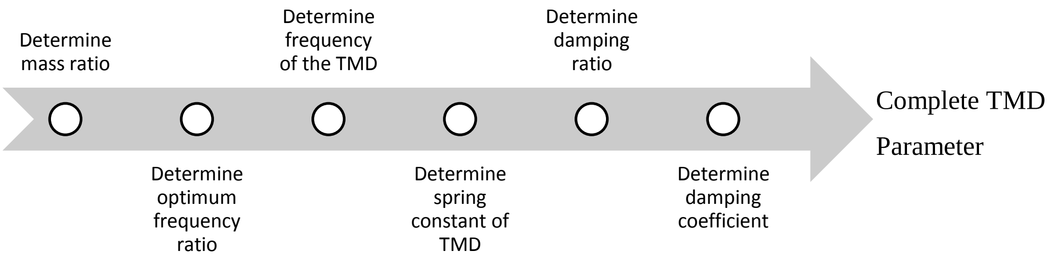
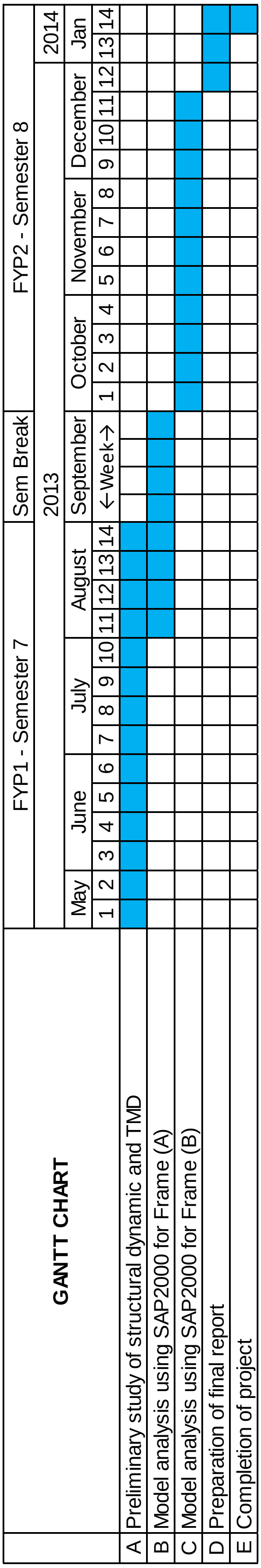
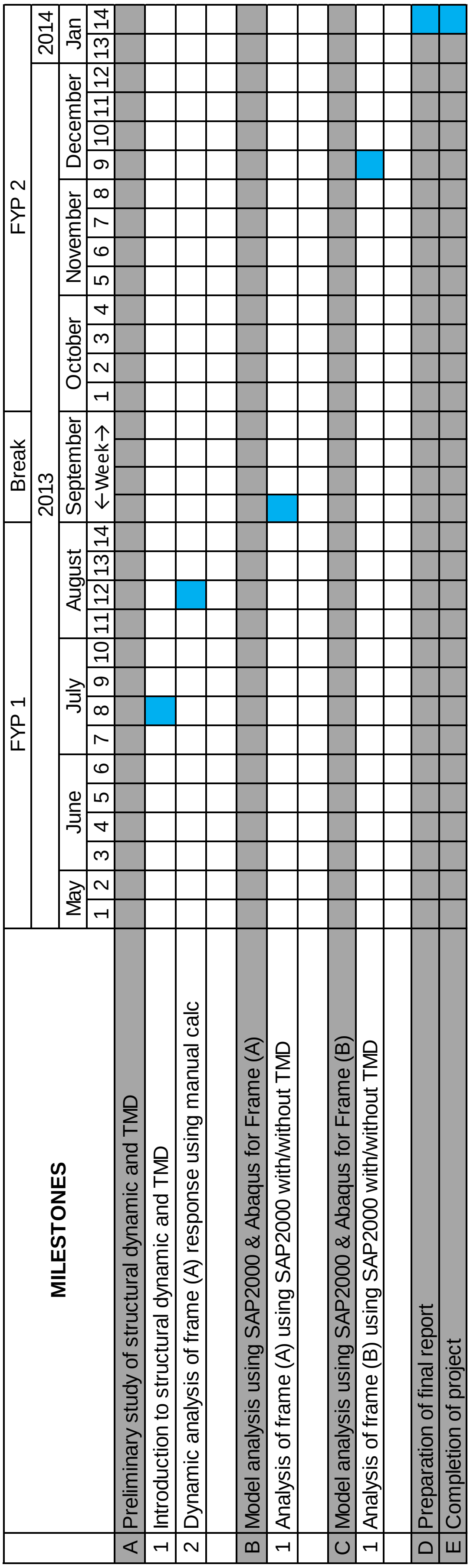


Figure 3.7 Simplified TMD design procedure

3.5 Gantt chart and Key Milestones





4 RESULT AND DISCUSSION

4.1 Dynamic Analysis of Frame A

Frame (A) each storey height is 3m

The column second moment of inertia is $7.618 \times 10^{-5} m^4$

The column young modulus $E = 2 \times 10^8\ kN/m^2$

Storey stiffness $k = 2 \times \frac{12 \times 2 \times 10^8 \times 7.618 \times 10^{-5}}{3^3} = 13543.1\ kN/m$

Dynamic Response

First mode

$$\omega = 0.618 \sqrt{\frac{k}{M}} = 0.618 \sqrt{\frac{13543.1}{50}} = 10.17\ rad/s$$

Frequency $f = 10.17/2\pi = 1.62\ Hz$

Period $T = \frac{1}{1.62} = 0.618\ s$

Second mode

$$\omega = 1.618 \sqrt{\frac{k}{M}} = 1.618 \sqrt{\frac{13543.1}{50}} = 26.62\ rad/s$$

Frequency $f = 26.62/2\pi = 4.24\ Hz$

Period $T = \frac{1}{5.29} = 0.235\ s$

SAP2000 output

Output cases	**Period (s)**	**Frequency (Hz)**	**Circular Frequency (rad/s)**
Mode 1	0.639471	1.5638	9.8256
Mode 2	0.246067	4.0639	25.534

Table 4.1 SAP2000 Dynamic Response Result of frame A

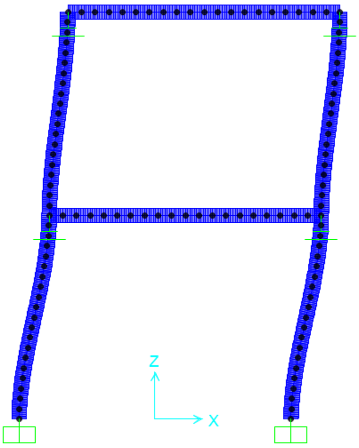


Figure 4.2 Mode 1 Characteristic Shape

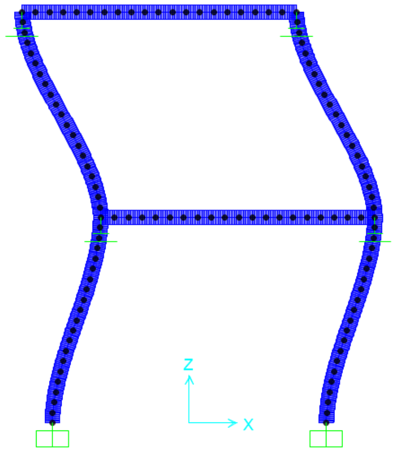


Figure 4.1 Mode 2 Characteristic Shape

Output cases	**Manual Calculation** **Period (s)**	**SAP2000** **Period (s)**	**Differences**
Mode 1	0.618	0.639471	+3.46%
Mode 2	0.235	0.246067	+4.68%

Table 4.2 Comparison between Manual Calculation and SAP2000 result

The result from manual calculation is from an idealize frame where some factors are left out of consideration especially the generalization of stiffness of the column. Whereas, result from SAP2000 which use finite element method can consider as more accurate. Comparison between the two result shown that it is acceptable to use SAP2000 as the tool in this study.

4.2 Time History Analysis of Frame A without TMD

A sinusoidal force with amplitude *300N* was applied to the Frame A at top floor with varying period.

4.2.1 Sinusoidal Force with period $T = 1\ s$

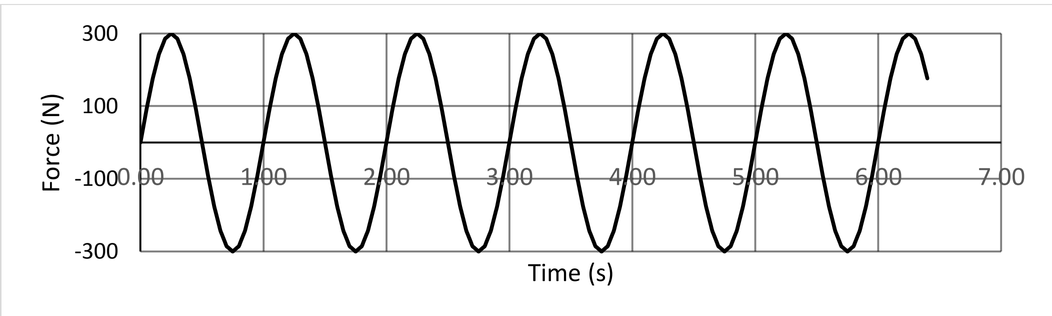


Figure 4.3 Characteristic wave of applied force with T=1s

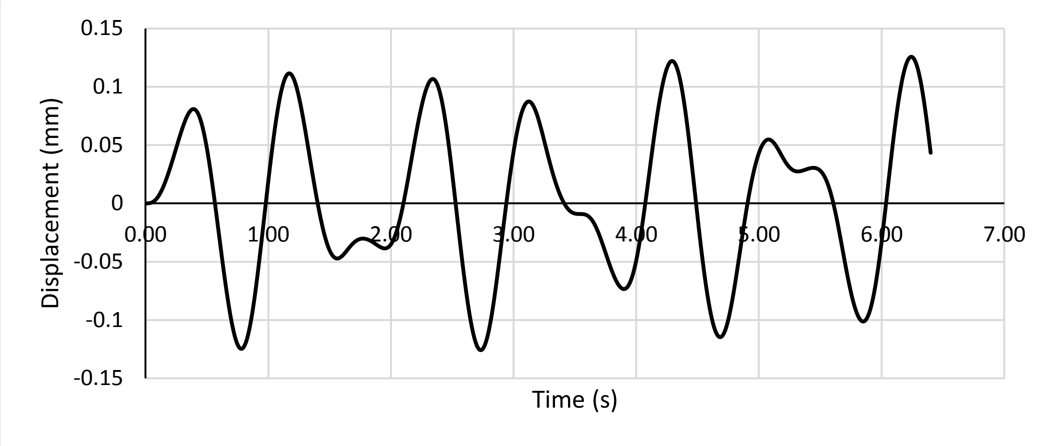


Figure 4.4 Displacement over Time of Frame A (Top floor) under applied force with T=1s

The applied sinusoidal force with T=1s, which is higher than the structure natural period of T=0.64s, does not cause resonant effect to Frame A. Figure 4.4 shown that the displacement of Frame A top floor is relatively low and fall between 0.13mm and -0.13mm over a timeframe of 7 seconds.

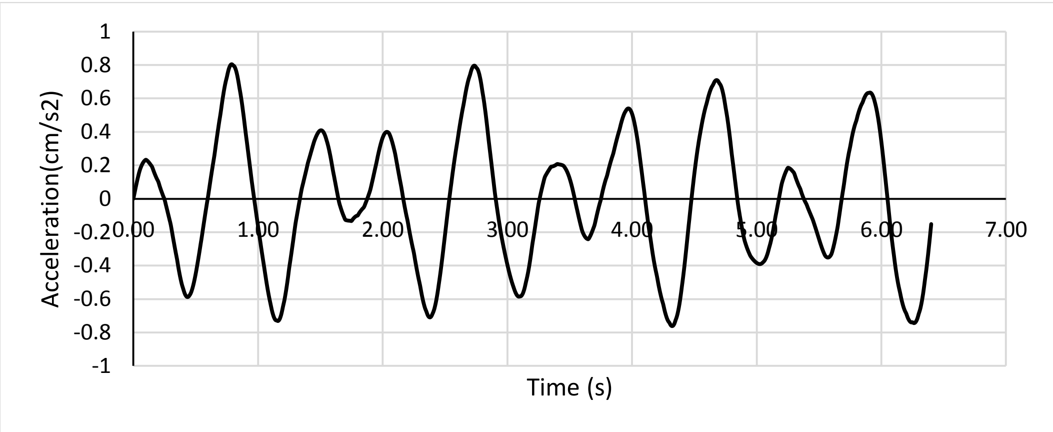


Figure 4.5 Acceleration over Time of Frame A (Top floor) under applied force with T=1s

Likewise, the acceleration also fell within a certain domain.

4.2.2 Sinusoidal Force with period T=0.64s

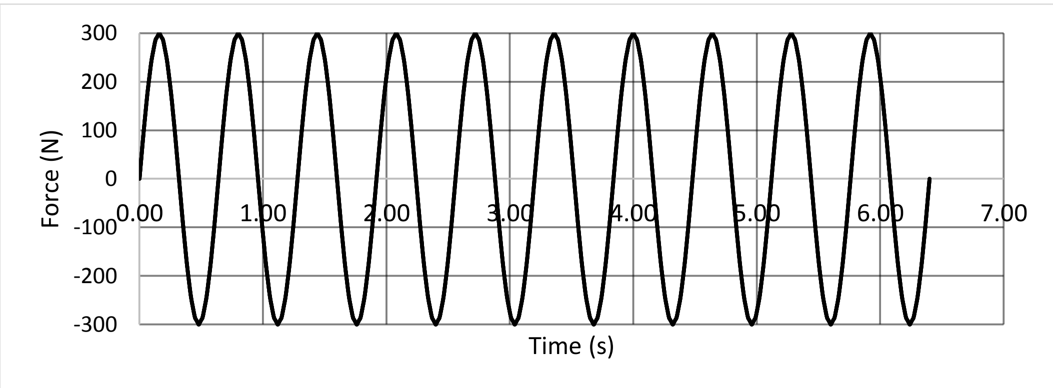


Figure 4.6 Characteristic wave of applied force with T=0.64s

The force applied in this case is exactly match the natural period of frame A.

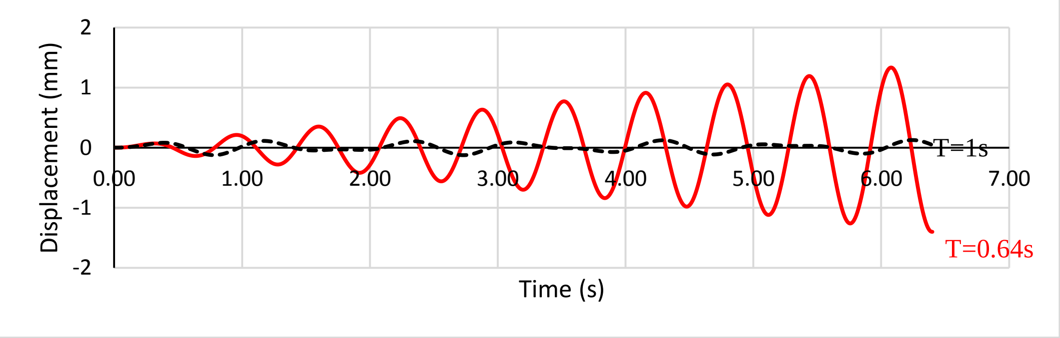


Figure 4.7 Displacement over Time of Frame A (Top floor) under applied force with T=0.64s

Figure 4.7 clearly show the effects of resonance caused by excitation with frequency matching the natural frequency of the structure. The displacement keep increasing overtime to over 1mm after 4.5s comparing to applied force T=1s which displaced only 0.05mm at the same moment.

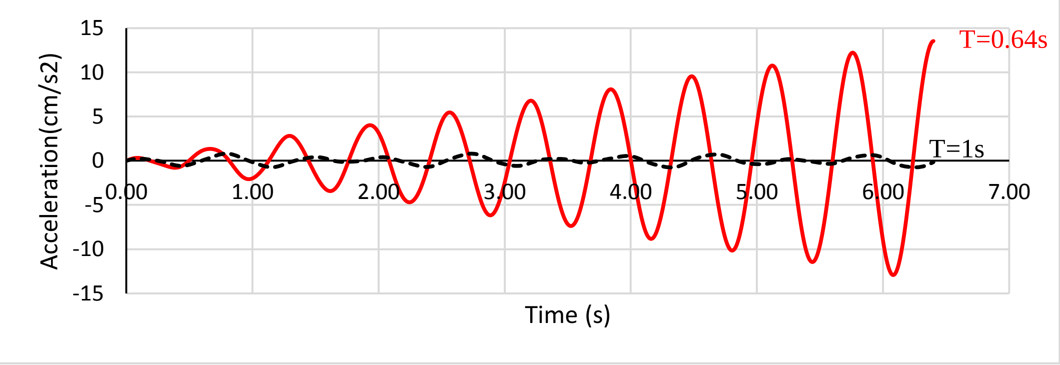


Figure 4.8 Acceleration over Time of Frame A (Top floor) under applied force with T=0.64s

Acceleration also become perceptible at more than 5 cm/s^2 after 2s and dangerously increase higher.

4.2.3 Sinusoidal Force with period T=0.25s

Frame A also have another fundamental period for mode 2 of the structure with T=0.25s. So Sinusoidal Force with period T=0.25s was applied to see its effects on the structure.

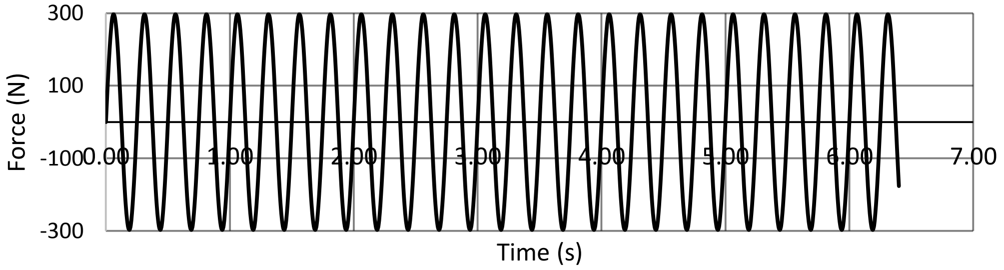


Figure 4.9 Characteristic wave of applied force with T=0.25s

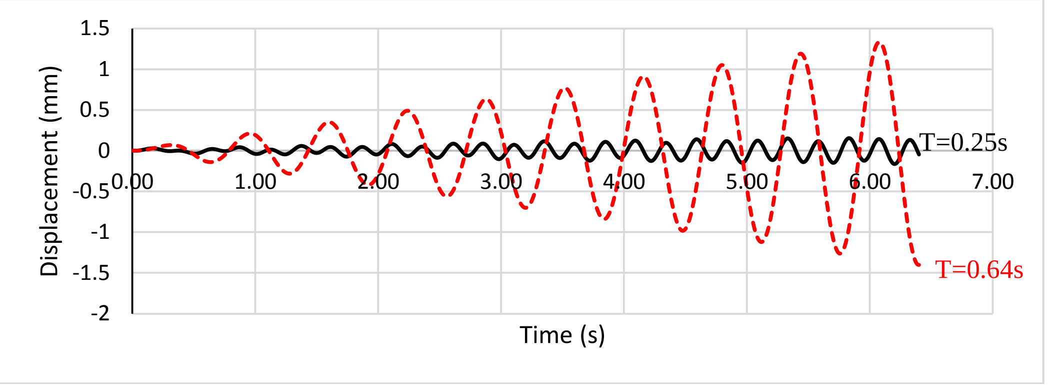


Figure 4.10 Displacement over Time of Frame A (Top floor) under applied force with T=0.25s

The displacement caused by excitation matching natural frequency of mode 2 of frame A is increasing but relatively small compare to the displacement cause by excitation matching natural frequency of mode 1.

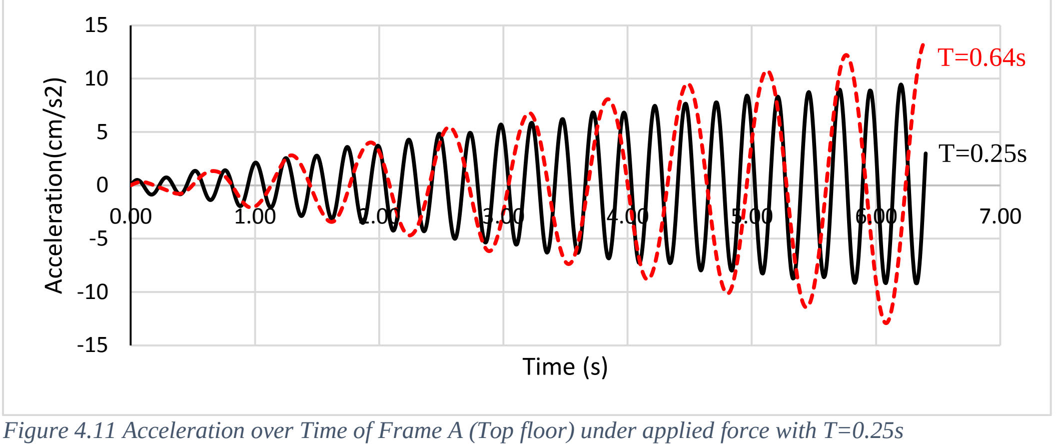


Figure 4.11 Acceleration over Time of Frame A (Top floor) under applied force with T=0.25s

The acceleration caused by this excitation is quite high mainly might due to its high frequency nature and the small size of the structure. This result show that it is adequate to only consider the natural frequency of mode 1 and design the TMD accordingly.

4.3 Time History Analysis of Frame A with TMD

4.3.1 TMD Parameter

Mass ratio chosen to be $\mu = 0.04$ so mass of TMD $m_d = 4000$kg

Optimum frequency ratio $\alpha_{opt} = 0.96154$

Frequency of damper $f_d = 1.5036\ Hz \rightarrow\ \omega_d = 9.45\ rad/s$

Spring stiffness constant $k = 357\ kN/m$

Optimum damping ratio $\zeta_{opt} = 0.1155$

Damping coefficient $c = 8.72\ kN.s/m$

The force applied is sinusoidal excitation matching the natural frequency of mode 1 T=0.64s.

4.3.2 Result

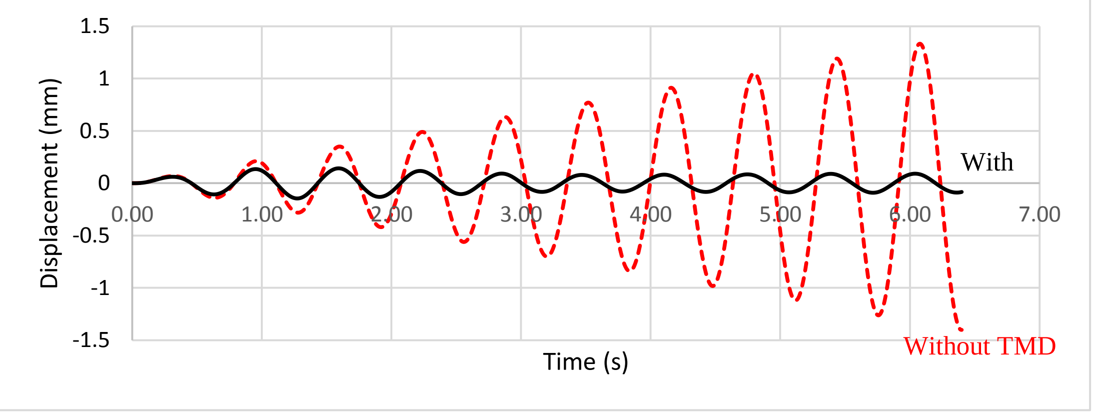


Figure 4.12 Displacement over Time of Frame A (Top floor) with TMD

The TMD has reduce the displacement of the frame under resonant force to within certain range which is very small compare to the displacement caused without TMD.

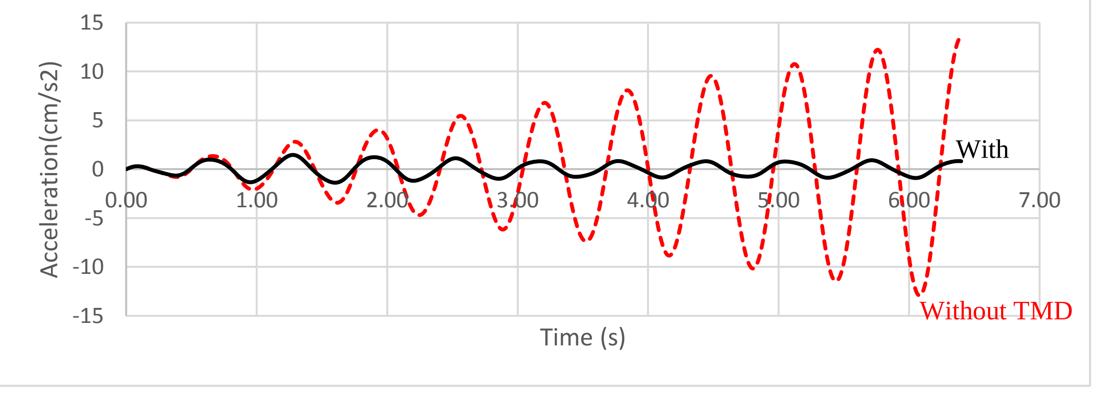


Figure 4.13 Acceleration over Time of Frame A (Top floor) with TMD

Similarly, the acceleration of frame A was maintain to a comfortable level with the help of TMD.

4.4 Dynamic Analysis of Frame B

SAP 2000 calculate the natural frequency of structure to be $f = 0.33272\ Hz$ for x-direction and $f = 0.38112\ Hz$ for y-direction.

Structure total mass is 9055.56 kN-s^2/m with participating modal mass of 81.169% = 7350.31 kN- s^2/m.

4.5 Time History Analysis of Frame B without TMD

A sinusoidal force with amplitude *60kN* was applied to the Frame B at top floor with varying period.

4.5.1 Sinusoidal Force with period $T = 1\ s$

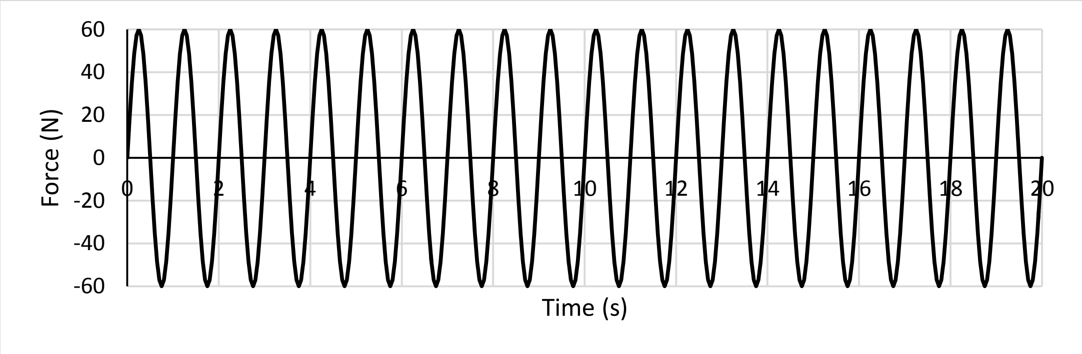


Figure 4.14 Characteristic wave of applied force with T=1s Frame B

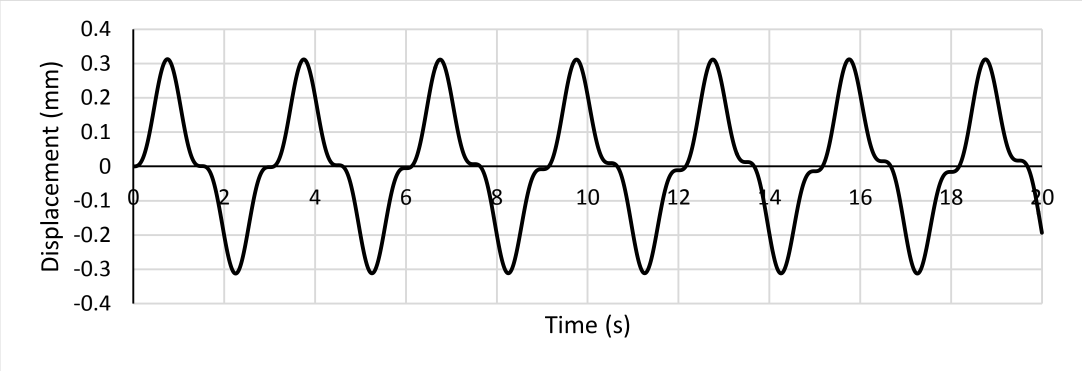


Figure 4.15 Displacement over Time of Frame B (Top floor) under applied force with T=1s

The applied sinusoidal force with T=1s, which is higher than the structure natural period of T=3.0055s, does not cause resonant effect to Frame B. Figure 4.15 shown that the displacement of Frame B top floor is relatively low and fall between 0.3mm and -0.3mm over a timeframe of 20 seconds. Acceleration also behave as expected.

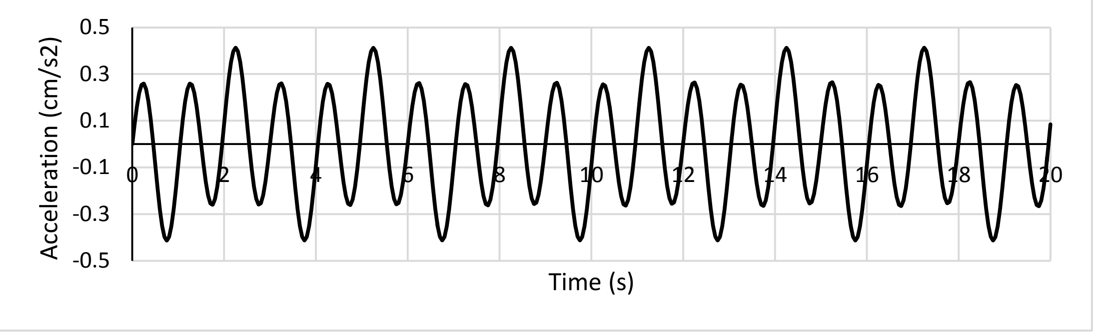


Figure 4.16 Acceleration over Time of Frame B (Top floor) under applied force with T=1s

4.5.2 Sinusoidal Force with period $T = 3.0055\ s$

The force applied in this case is exactly match the natural period of frame A.

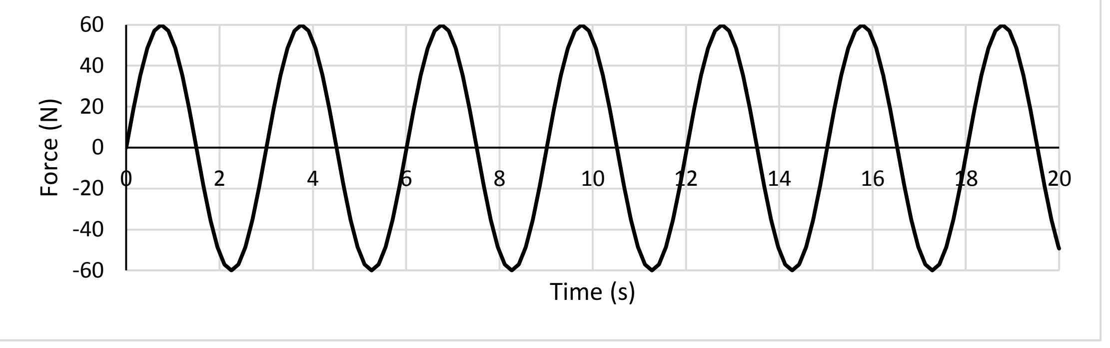


Figure 4.17 Characteristic wave of applied force with T=3.0055s Frame B

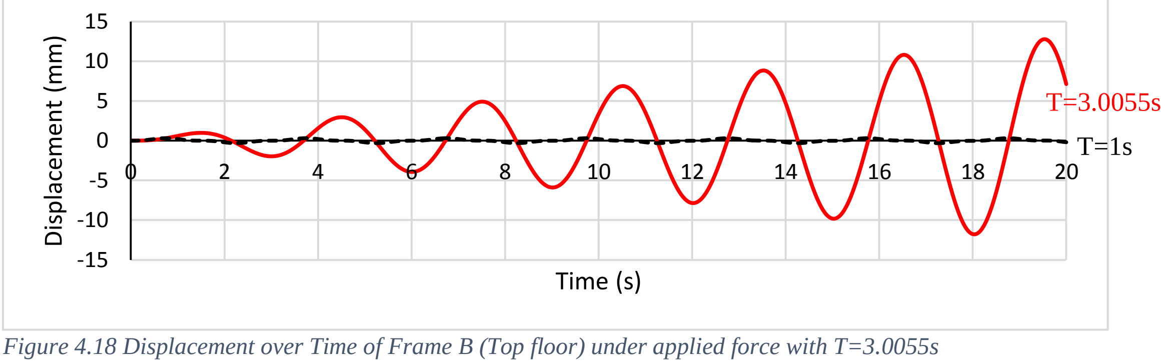


Figure 4.18 Displacement over Time of Frame B (Top floor) under applied force with T=3.0055s

Similar to characteristic of frame A Figure 4.18 clearly show the effects of resonant caused by excitation with frequency matching the natural frequency of the structure. The displacement keep increasing overtime.

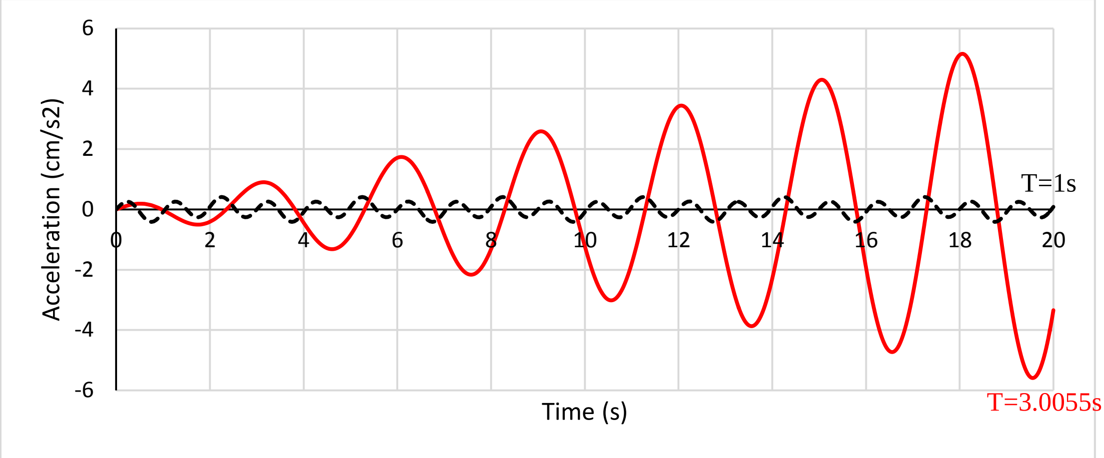


Figure 4.19 Acceleration over Time of Frame B (Top floor) under applied force with T=3.0055s

As observed, acceleration also increased to a perceptible level.

4.6 Time History Analysis of Frame B with TMD

4.6.1 TMD Parameter

Mass ratio chosen to be $\mu = 0.004$ so mass of TMD $m_d = 30000$kg

Optimum frequency ratio $\alpha_{opt} = 0.99601$

Frequency of damper $f_d = 0.3313944\ Hz \rightarrow\ \omega_d = 2.0822\ rad/s$

Spring stiffness constant $k = 130.07\ kN/m$

Optimum damping ratio $\zeta_{opt} = 0.03865$

Damping coefficient $c = 4.83\ kN.s/m$

The force applied is sinusoidal excitation matching the natural frequency of mode 1 T=3.0055s.

4.6.2 Result

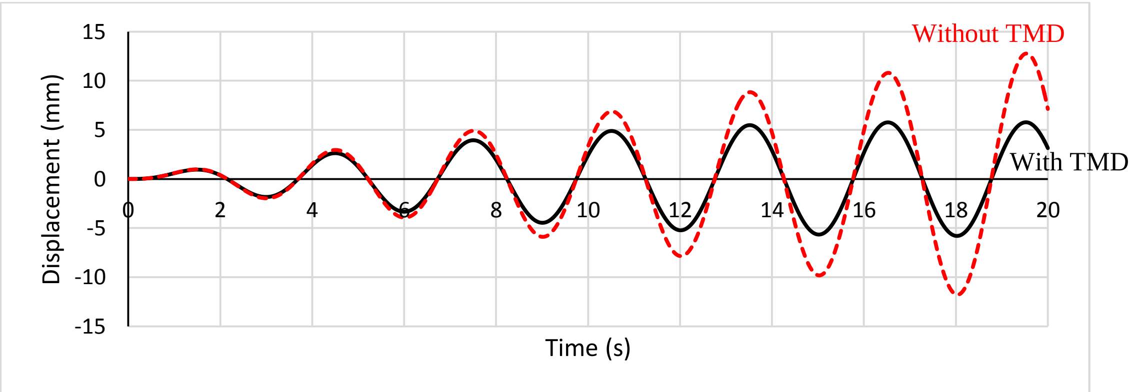


Figure 4.20 Displacement over Time of Frame B (Top floor) with TMD

The TMD has reduce the displacement of the frame under resonant force to within certain range which is very small compare to the displacement caused without TMD.

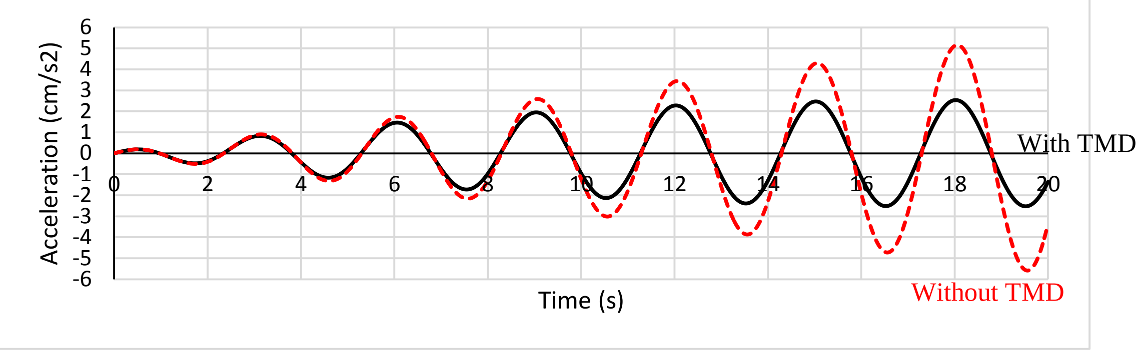


Figure 4.21 Acceleration over Time of Frame B (Top floor) with TMD

The acceleration of frame B was maintain to a comfortable level with the help of TMD as expected.

5 CONCLUSION

The result from manual calculation is from an idealize frame where some factors are left out of consideration especially the generalization of stiffness of the column. Whereas, result from SAP2000 which use finite element method can consider as more accurate. Comparison between the two result shown that it is acceptable to use SAP2000 as the tool in this study.

Time History analysis on frame A shown that periodic force with frequency matching the natural frequency of the structure create the resonant effect causing excessive displacement and acceleration. This is further confirmed in the analysis on frame B which shown similar characteristic of resonant effect. TMD application on frame A using SAP2000 successfully mitigate the resonant of frame A to an acceptable limit. Frame B vibration problem was also mitigated by using the TMD characteristic calculate by Den Hartog’s optimization criteria.

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