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APPLICATION OF TUNED LIQUID DAMPERS TO MITIGATE WIND-INDUCED TORSIONAL MOTION

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APPLICATION OF TUNED LIQUID DAMPERS TO MITIGATE WIND-INDUCED TORSIONAL MOTION

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Abstract

The tuned liquid damper (TLD) is a proven and an increasingly popular auxiliary device for mitigating the dynamic effects induced by wind loading on tall buildings. During a dynamic loading event, the water inside a TLD will slosh against the end walls of the tank, thereby imparting a force approximately anti-phase to the motion of the building. The current study uses a multi-modal TLD system to reduce the resonant torsional responses of a real high-rise building. The building is sensitive to torsion in the first two vibration modes; therefore, a unique TLD system is designed to damp these two modes by displacing the tanks away from the center of mass of the building. The TLD system is capable of reducing the serviceability responses to an acceptable level. In addition, the current study demonstrates the possible reduction in wind loading experienced by the building. The reduced wind loading leads to a 16.9% reduction in the cost of steel reinforcement in the concrete shear walls. Furthermore, the robustness of the TLD system is evaluated and practical TLD design issues are discussed.

Keywords: torsion, high-rise building, tuned liquid damper, wind loading, high frequency force balance, serviceability, practical

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Nomenclature

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a	modal mixing factor for the GWF
A	amplitude of excitation
b	dispersion parameter for a Type I extreme-value distribution
b_w	width of the tank
B	projected width of the building
C	correction factor for the mode shapes of the building
c^*	generalized damping coefficient of the building
c_{TLD}	generalized damping coefficient of the TLD
e	eccentricity between CM and CC of the building
f	modal frequency of the building
f_s	structural vibration frequency
f_w	sloshing frequency of the water inside the tank
f_{TLD}	equivalent amplitude-dependent TMD frequency
$\hat{F}$	force or torque for the ESWL of the building
$F^*(t)$	generalized wind force for the building
g	acceleration of gravity
h	storey height
h_w	height of water in the tank
H	height of the building
I	storey mass moment of inertia
k^*	generalized stiffness of the building
k_{TLD}	generalized stiffness of the TLD
L_w	length of the tank
L_x	distance between CM and the center of the tank in the X direction
L_y	distance between CM and the center of the tank in the Y direction
m	storey mass
m^*	generalized mass of the building
m_1	mass of water inside the tank contributing to the fundamental sloshing mode

m_o	non-participating water mass in the tank
m_p	generalized water mass for a TLD
m_s	total mass of the structure
m_{TLD}	equivalent amplitude-dependent TMD mass
M_p	total generalized water mass for the TLD system
M_w	total physical water mass for the TLD system
$\bar{M}$	mean base moment
$M_B(t)$	background base moment
$\widetilde{M}_B$	peak hourly background base moment
$\widehat{M}_Q$	peak hourly quasi-steady base moment
$M_R(t)$	resonant base moment
$\widetilde{M}_R$	peak hourly resonant base moment
M_x	base moment of the building in the X direction
M_y	base moment of the building in the Y direction
M_θ	base torque of the building
N	number of intervals for a Type I extreme-value distribution
p	trapezoidal pressure shape for ELS
$P(X \leq x)$	probability that a given value of x will not be exceeded
R^2	measure of how well a regression line fits a set of data
$R_{No\ TLD}$	response without a TLD system
$R_{With\ TLD}$	response with a TLD system
$\hat{S}_Q$	peak hourly quasi-steady base shear
t	time
T	base torque of the building
u	location parameter for a Type I extreme-value distribution
z	elevation of the building
α	angle of wind incidence on the building
α	angle of wind incidence on the building
α_p	proportion of the water mass contributing to the fundamental sloshing mode
γ	denotes the direction (equals x, y, or θ)

ζ	damping ratio
ζ_{TLD}	equivalent amplitude-dependent TMD damping ratio
μ	TLD mass ratio of the TLD to the structure
ξ_s	generalized displacement of the structure
$\dot{\xi}_s$	generalized velocity of the structure
$\ddot{\xi}_s$	generalized acceleration of the structure
ξ_{TLD}	generalized displacement of the TLD
$\dot{\xi}_{TLD}$	generalized velocity of the TLD
$\ddot{\xi}_{TLD}$	generalized acceleration of the TLD
φ	mode shape of the building
$\bar{\chi}$	effective load shape for the mean base moment
$\tilde{\chi}_B$	effective load shape for the peak hourly background base moment
$\tilde{\chi}_R$	effective load shape for the peak hourly resonant base moment
Φ	amplitude modification factor
Ψ	reduction in the building response from installing a TLD system
Ω	tuning ratio of the TLD to the structure

List of Subscripts

B	background
eff	effective
i	storey number
j	mode number
opt	optimal
Q	quasi-steady
R	resonant
X	X direction for the building
Y	Y direction for the building
θ	torsional direction for the building

List of Abbreviations

2DOF	Two Degree-of-Freedom
AMF	Amplitude Modification Factor
BLUE	Best Linear Unbiased Estimator
BLWTL	Boundary Layer Wind Tunnel Laboratory
BBM	Base Bending Moment
BM	Base Moment
CC	Center of Coordinates
CF	Correction Factor
CI	Confidence Interval
CM	Center of Masses
DOF	Degree-of-Freedom
DVA	Dynamic Vibration Absorber
ELS	Effective Load Shape
ESWL	Equivalent Static Wind Load
FOS	Factor of Safety
GLF	Gust Loading Factor
GWF	Generalized Wind Force
HFFB	High Frequency Force Balance
MRF	Moment Resisting Frame
MTLD	Multiple Tuned Liquid Damper
NBCC	National Building Code of Canada
RMS	Root-Mean-Square
SDOF	Single Degree-of-Freedom
SRSS	Square Root of the Sum of Squares
STLD	Single Tuned Liquid Damper
TLD	Tuned Liquid Damper
TMD	Tuned Mass Damper
TSD	Tuned Sloshing Damper

CHAPTER 1

Introduction

1.1 General Overview

Excessive motions in buildings cause occupants to become uncomfortable and nervous. This is particularly detrimental to the tenants and ultimately the owner of the building, with respect to financial considerations. Serviceability issues, such as excessive accelerations and inter-storey drifts, are more prevalent today due to advancements in the structural systems, strength of materials, and design practices. These factors allow buildings to be taller, lighter, and more flexible, thereby exacerbating the impact of dynamic responses. For many tall buildings, a dynamic vibration absorber (DVA) is required to ensure that occupants, tenants, and owners remain comfortable. A DVA effectively increases the damping of a structure by dissipating energy when the device is subjected to movement. An effective and reputable DVA is the tuned liquid damper (TLD). This thesis specifically focuses on the efficiency of the TLD in reducing the motions of a lateral-torsional coupled high-rise building. In addition, the current study investigates the use of TLDs, not only for serviceability requirements, but also for strength design. This is through the reduction of the design wind loads acting on buildings due to the addition of TLDs.

1.2 Passive Dynamic Vibration Absorbers

A passive DVA is a device that is capable of controlling structural vibrations without obtaining feedback from motion sensors installed on the structure or requiring a power source. If feedback or a power source is provided, the DVA is classified as active. A

passive DVA does not require any feedback from the structure – it simply reacts to the dynamic motions of the structure. The two most common passive DVAs are the tuned mass damper (TMD) and the TLD. Both dampers interact with a structure by essentially adding an auxiliary mass, spring, and dashpot system to the main structural system.

1.2.1 Tuned Mass Damper

A TMD is simply a large mass held in place by springs and dashpots, which is attached to the structure and permitted to move, or oscillate, by a predetermined maximum displacement. The mass and stiffness of the spring are selected to achieve an oscillation frequency that is approximately equal to the structural frequency of the mode that is to be suppressed. During dynamic excitation, the TMD oscillates against the motions of the building, thereby imparting an inertial force that is approximately anti-phase to the structural motion. This will mitigate the structural responses, such as deflections and accelerations.

The concept of dissipating energy by use of a TMD dates back to 1909; however, McNamara (1977) first applied this concept to buildings as a means to reduce wind-induced structural responses in the elastic range of behaviour. Luft (1979) advanced this concept by determining the optimal frequency and effective damping required for any building based on the associated dynamic characteristics. Some of the first applications of a TMD are in the Citicorp Center in New York City and in the John Hancock Tower in Boston (McNamara, 1977). A famous example is the CN Tower in Toronto, which has two TMDs to suppress the second and fourth structural modes of vibration (Kwok and Samali, 1995). These applications are specific to lateral motion of buildings under wind loading.

Torsional motion presents a different challenge for TMD design, which is overcome by using an eccentric mass model. Wind tunnel testing on aeroelastic models with a scaled TMD demonstrates the ability of a TMD to reduce the torsional motion (Xu et al., 1992). Furthermore, Singh et al. (2002) recognized a need to develop an optimal approach for torsional TMD design when buildings are subjected to bi-directional seismic loading. A similar study by Ueng et al. (2008) demonstrated a technique to determine the optimal parameters for several TMDs tuned to different structural frequencies under bi-derectional seismic loading. The results indicate that a multi-modal TMD placed away from the center of mass of the building will effectively reduce the structural motion. Moreover, as the eccentricity of the TMD increases, the effectiveness increases because this effectively increases the contributing inertial mass.

1.2.2 Tuned Liquid Damper

A TLD is a rigid tank that is partially filled with a liquid – usually water. The sloshing frequency of a rectangular TLD depends on the length of the tank and the height of the water. Similar to the TMD, the TLD sloshing frequency is set to match the structural frequency for the mode that is to be suppressed. When dynamically excited, the free surface of the liquid sloshes against the end walls of the tank, thereby imparting an inertial force approximately anti-phase to the structural motion. This structure-damper interaction is similar for both a TLD and a TMD. The main difference between the two dampers is in the determination of the mass, stiffness, and damping ratio for the auxiliary system attached to the structure.

The sloshing action inside the tank causes the TLD to behave nonlinearly (Tait et al., 2005a). Installing slat screens inside the tank mitigates the nonlinear behaviour and

increases the inherent damping. The inherent damping is significantly lower than the optimal value without the screens (Fediw et al., 1995). Despite the improved performance and reduced nonlinearities from installing slat screens, the mass, stiffness, and damping ratio for the auxiliary system remain nonlinear (Tait et al., 2004a). The simplest and most effective procedure to assign a mass, stiffness, and damping ratio to the complicated sloshing behaviour of the water is to transform the TLD into an equivalent amplitude-dependent TMD. Sun et al. (1995) formulated this concept, which involves equating the energy dissipated by the TLD to the energy dissipated by an equivalent TMD for a particular amplitude of excitation. Tait et al. (2004a) modified the concept to incorporate slat screens. The result is a description of the TLD through an equivalent amplitude-dependent TMD mass, frequency, and damping ratio. The frequency is used to calculate a stiffness, which in conjunction with the mass and damping ratio defines the auxiliary system attached to the structure. Tait et al. (2004b) verified the equivalent amplitude-dependent TMD model and demonstrated the robustness of the TLD. Furthermore, Tait et al. (2005b) conducted shake table experiments on TLDs under bi-directional excitation. The results demonstrated that the 2D TLD can be analysed as two independent 1D TLDs because the energy dissipation is uncoupled in the orthogonal directions.

The first DVA utilising the sloshing motion of a liquid to dissipate energy is known as the nutation damper, which is a torus shaped container (Modi et al., 1990). The nutation damper is popular in Japan for reducing the wind-induced vibrations of airport control towers (Fujii et al., 1990; Tamura et al., 1992; Tamura et al., 1995). One of the first TLD applications to a tall building is in the Shin Yokohama Prince Hotel in Japan (Wakahara et al., 1992). Many other high-rise applications have followed, including the use of

rectangular TLDs in the One King West Tower in Toronto, (Hasan, 2008). These cases utilise TLDs to dissipate energy from wind loading in the sway directions, which has a relatively small excitation amplitude. Reed et al. (1998) investigated the effect of large amplitude excitations on tuned liquid dampers. The results indicated that the nonlinear sloshing motion increases the effectiveness and robustness of the TLD in controlling structural motions under large amplitude excitations. Banerji et al. (2000) confirmed this by demonstrating the ability of the TLD to perform effectively during earthquake loading – considered a large amplitude excitation.

Fujino and Sun (1993) proposed a technique to enhance the effectiveness of a TLD system by tuning multiple tanks to slightly different sloshing frequencies around the optimal frequency. This promotes a robust multiple tuned liquid damper (MTLD) system that will behave properly despite an off tuning because the system captures a larger frequency bandwidth. Li et al. (2004) proposed a similar technique, however, with several MTLD systems tuned to different structural modes of vibration. Through shake table experiments, the results demonstrated that the first several dominant modes can be controlled with MTLDs for high-rise buildings. In addition, Rahman (2007) demonstrated the effectiveness of an MTLD system at reducing the structural damage in reinforced concrete frames caused by strong earthquakes. In fact, the MTLD was capable of eliminating the concrete crushing and steel yielding of many beam-column connections in the frame, which the single tuned liquid damper (STLD) was unable to accomplish.

1.3 Wind Loading

Accurately assessing the wind acting on a structure and the associated dynamic effects is essential for proper and safe structural design. To understand the wind loading on a

structure, wind tunnel testing is indispensable. This is especially true for tall buildings because they are more susceptible to the dynamic effects induced by fluctuating winds, which are not fully accounted for in building codes. A significant dynamic contribution raises the issue of meeting serviceability criteria, such as excessive accelerations.

The threshold for when accelerations become excessive is difficult to quantify because every building occupant will have a different perception threshold. In addition, many factors influence this threshold, such as body orientation, body movement, body posture, expectancy of sway, and visual cues (Chen and Robertson, 1972). Hansen et al. (1973) conducted surveys of the occupants in two tall buildings after a strong wind storm. The conclusion is that the perception threshold follows a lognormal distribution from which a limit can be chosen based on the number of occurrences that are acceptable in any given year. Tallin and Ellingwood (1984) proposed a root-mean-square (RMS) acceleration serviceability limit of 10 milli-g for the average duration of one tenancy (8 years in office buildings). Furthermore, Melbourne and Palmer (1992) developed peak acceleration criteria, instead of RMS, for buildings that exhibit complex lateral-torsional motions. The allowable peak hourly accelerations for a 10-year return period established by the Boundary Layer Wind Tunnel Laboratory (BLWTL) are 10 to 15 milli-g for residential buildings, 15 to 20 milli-g for hotels, and 20 to 25 milli-g for office buildings (Isyumov, 1994). Notably, a TLD is found to perform optimally when the peak hourly acceleration for a 10-year return period is within 12 to 22 milli-g (Tait et al., 2008b).

1.3.1 High Frequency Force Balance Technique

To obtain serviceability and strength design values, the high frequency force balance (HFFB) testing technique is a popular and cost effective approach. Tschanz and

Davenport (1983) first developed the HFFB technique as a means to simplify dynamic force measurements. The technique involves using a five component base balance, which measures the base shear and moment in two directions and the base torque. A rigid lightweight foam model, representing the test building, is placed atop the base balance.

The basis of the HFFB technique is to establish a generalized wind force (GWF) acting on the building from the base measurements. Yip and Flay (1995) updated the GWF formulation to account for coupled three-dimensional modes. However, in forming the GWF, the following assumptions are still required: the mode shapes in the sway directions are linear and the torsional mode shape is constant (Tschanz and Davenport, 1983). Generally, the mode shapes of high-rise buildings are nonlinear. Consequently, correction factors (CFs) are required to adjust the actual mode shapes to match the assumed shapes. Researchers have developed various approaches to calculate the CFs; however, the consensus is that the CFs depends on the load shape, which is approximated by using the power law exponent of both the mode shape and mean wind speed profile (Xu and Kwok, 1993; Zhou et al., 2002; Holmes et al., 2003; Lam and Li, 2009).

1.3.2 Equivalent Static Wind Loads

Equivalent static wind loads (ESWLs) are a set of response-specific pressures, forces, or torques such that when statically applied to a structure the chosen peak response matches the peak response induced by the fluctuating winds. Davenport (1967) pioneered a procedure to establish a static loading for a structure, which produced the peak top deflection by multiplying the mean deflection by a gust loading factor (GLF). This procedure has since been updated by many researchers to represent the resonant component of the wind loading more accurately. Boggs and Peterka (1989) first proposed

a method to distribute the resonant load proportionally by the mode shapes and masses of the building. Secondly, Holmes (2002) accounted for modes that are higher than the fundamental, which is important for buildings that are dynamically sensitive. Finally, Chen and Kareem (2005b) proposed a method to account for three-dimensional coupled modes.

A significant contribution by Zhou and Kareem (2001) modified the GLF method to be based on the base bending moments (BBMs), instead of top deflection, because it represents the cantilever action of tall buildings more realistically. In addition, the BBM based method provides a solid framework for use in conjunction with the HFFB testing technique (Chen and Kareem, 2005a).

1.4 Impetus and Research Objectives

There is a growing need for innovative and effective techniques to reduce the serviceability responses of increasingly taller, lighter, and more flexible buildings. The use of TLDs is still a relatively new concept compared to TMDs; however, they are promising devices for controlling the dynamic responses of high-rise buildings. The implementation of TLDs in real buildings is self-promoting the benefits and effectiveness of using this cost effective device. The motivation of this research is from the need to develop a TLD system for use in torsionally sensitive buildings because currently there is no published work in this area. A real building exhibiting highly coupled lateral-torsional motion is selected because the responses exceed the serviceability criteria. In addition, there are other potential benefits from installing a TLD system, including a reduced resonant loading on the structural members of the building. There is no published work on the effect of a DVA in reducing the resonant wind load on a building. Consequently, a

TLD system has the potential to reduce the strength requirements of a building, thus resulting in a material cost savings. To investigate these issues, the current study has the following objectives.

1) Develop a design procedure for a multi-modal TLD system that is specifically intended to reduce lateral-torsional coupled motion in high-rise buildings.
2) Develop a multi-modal analysis method, which is capable of solving the responses and ESWL of a structure-TLD system subjected to an excitation force calculated from HFFB test data.
3) Using this analysis method, determine the effectiveness and robustness of a TLD system at mitigating the serviceability responses and reducing the ESWL.
4) Using the reduced ESWL, determine the allowable reduction in materials achieved by installing TLDs on a high-rise building.

1.5 Organization of Thesis

This thesis is written in the integrated-article format. As a result, each chapter will include a separate bibliography. Since the chapters are written as standalone documents, overlap does occur between each chapter – primarily in the discussion of the test building and the TLD system configurations.

Chapter 2 reports on a real building that exhibits a torsional sensitivity and exceeds the serviceability criteria. Three unique multi-modal TLD systems are designed specifically to mitigate the torsional response of the building. A procedure is developed to analyse a structure-TLD system using HFFB test data from the BLWTL at the University of Western Ontario. The effectiveness of the unique TLD systems is investigated. In addition, a practical parametric study is conducted to determine the

robustness of the systems in reducing the serviceability responses.

Chapter 3 reports on the same torsionally sensitive building and the three unique TLD systems. The framework for calculating an ESWL using HFFB test data for a building with a TLD system is established. The ability of each TLD system to reduce the design ESWL is demonstrated for multiple damping cases. Similar to chapter 2, a parametric study is conducted on the TLD system to determine the effect of varying practical parameters on the base shears and moments.

Chapter 4 uses the reduced ESWLs from chapter 3 to design the shear walls of the torsionally sensitive building, with and without a TLD system installed. The chapter reports on the difference in steel reinforcement required for the concrete shear walls, such that both design cases have a factor of safety of one for an efficient design. The cost savings associated with the decreased reinforcement requirement is included.

Finally, chapter 5 summarizes the research and discusses the conclusions of the current study. Furthermore, the chapter includes recommendations for future studies.

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CHAPTER 2

Using Tuned Liquid Dampers to Control Wind-Induced Vibrations of a Torsionally Coupled Building

2.1 Introduction

Recent trends show that buildings are taller and more flexible, use lighter materials, and have innovative structural systems and less damping. This trend causes buildings to become more susceptible to dynamic loading, especially for those having complex shapes where torsion becomes an issue. A torsional sensitivity arises when a building has a complex shape, which leads to a significant eccentricity between the center of mass and center of rigidity. Combing a large eccentricity with a torsional stiffness that is less than the lateral stiffness, results in dominant torsion modes. This configuration leads to excessive motions when strong winds or earthquakes dynamically excite the building. To mitigate the motions to an acceptable level, the implementation of a secondary damping system is necessary. An effective passive dynamic vibration absorber (DVA) is the tuned liquid damper (TLD), which modifies the frequency response characteristics of the structure.

A TLD is a rigid tank partially filled with a liquid, usually water. The TLD sloshing frequency is tuned to the frequency of a specific mode of the structure that requires control. During dynamic excitation, the liquid will slosh against the walls of the tank. This sloshing motion imparts inertial forces approximately anti-phase to the dynamic excitation, thus reducing the structural motion. The advantages of using tuned liquid dampers (TLDs) are that they have low installation and maintenance costs, have an easily adjustable tuning frequency, can operate under a wide range of excitation amplitudes, and

are applicable for existing structures. A disadvantage is that space requirements can be high in order to achieve an adequate mass of water.

The first damper utilising liquid sloshing to dissipate energy is the nutation damper, which is a disc shaped container (Modi et al., 1990). Kareem (1990) applied the same energy dissipation concept to a rectangular tank called a tuned sloshing damper (TSD), which is generally much larger than a nutation damper. However, the inherent damping through viscous dissipation in the boundary layers of the liquid in the TSD is an order of magnitude less than optimal. Research has shown that using lattice screens will increase the inherent damping to an optimal level (Fediw et al., 1995; Warnitchai and Pinkaew, 1998). This modern form of the TLD introduces a complex liquid motion through the screens that requires difficult computation to predict the sloshing behaviour. Appropriate linearization assumptions were made to develop a technique to transform the TLD into an equivalent tuned mass damper (TMD), as a means to simplify the analysis (Sun et al., 1995). The technique equates the energy dissipation of the tuned liquid damper to an equivalent single degree-of-freedom (SDOF) TMD at a specific amplitude of excitation. The nonlinear TLD properties vary with the excitation amplitude; therefore, the TLD is represented as an SDOF with an amplitude-dependent set of equivalent TMD properties. Despite the dependency on amplitude, a TLD remains effective over a wide range of excitations, varying from small (wind) to large (earthquake) (Reed et al., 1998).

Tait et al. (2005a) developed a sophisticated numerical model to predict the sloshing motion of the TLD equipped with slat screens. Using this numerical model, Tait et al. (2004a) developed an improved equivalent TMD method to determine the amplitude-dependent properties of a TLD equipped with slat screens. Furthermore, Tait et al. (2004b) demonstrated that a TLD is an effective and robust DVA for one-dimensional

excitation. Tait et al. (2005b) extended the same conclusions for a TLD designed to dissipate energy in two orthogonal directions.

Despite the significant research conducted on tuned liquid dampers, the practical applications have been limited to reducing the lateral motion of buildings. In terms of serviceability criteria, the torsional vibration of a building can cause excessively high corner accelerations and displacements. Tuned mass dampers (TMDs) have been demonstrated to reduce the torsional behaviour of buildings when they are placed away from the center of rigidity, thereby acting as an eccentric mass that works against the motion of the building (Singh et al., 2002; Tse et al., 2007; Ueng et al., 2008; Xu et al., 1992). A TMD behaves similarly to a TLD by means of exerting an inertial force that opposes the motion; therefore, a TLD should also be capable of reducing torsional motions by implementing the eccentric mass model. The research discussed in this chapter assesses the ability of a TLD system to reduce the lateral-torsional coupled motion of a building.

Research has shown that using a multiple tuned liquid damper (MTLD) with a distributed tuning ratio over a range of frequencies around the fundamental structural frequency leads to a more effective and robust system (Rahman, 2007). Li et al., (2004) extended this concept to tuning the TLD or TMD system to multiple structural modal frequencies, which is especially useful for closely spaced fundamental frequencies. To achieve an optimized system, previous studies have shown that it is advisable to employ dampers that are tuned to the first few dominant modes of the building (Koh et al., 1995). This current study is conducted on an irregular high-rise building where the first two modes contribute significantly to the torsional response of the building; therefore, a multi-modal TLD system is implemented.

This study aims to show the effectiveness and robustness of the TLD in reducing the coupled lateral-torsional motion of a high-rise building under wind loading. The building was tested at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario using the high frequency force balance (HFFB) technique. The results proved that the building exceeds serviceability criteria; therefore, the impetus of the research came from the need to reduce the top floor accelerations to an acceptable level. Three unique TLD systems (each with different masses of water) are designed for the building. The structure-TLD systems are subjected to wind loading from the BLWTL data and are numerically solved to demonstrate the ability of the TLD system to reduce the structural motion. The better system is then chosen for a parametric study. Three practical parameters are varied to investigate the robustness of the TLD system: the height of water inside the tanks, the eccentricity of the tanks, and the structural modal frequencies.

2.2 Description of the Test Building

A 50-storey irregular reinforced concrete building with a height of 161.7 meters is considered in this study. The building has an L-shaped floor plan, which introduces a significant eccentricity between the center of mass and the center of rigidity, thereby presenting a torsional sensitivity. The lateral load resisting elements are shear walls, primarily near the center of the floor plan, and moment resisting frames, located along two exterior faces of the building. This exacerbates the torsional sensitivity because the torsional stiffness is significantly lower than the lateral stiffness. The stiffness disparity and non-coinciding center of mass and center of rigidity lead to highly coupled lateral-torsional action.

The coupled action is confirmed from a modal analysis conducted on the building to investigate the fundamental vibration characteristics. Figure 2.1 illustrates the first three vibration mode shapes that have corresponding periods of 6.30, 5.30, and 3.78 seconds. The torsion mode shapes are multiplied by the overall radius of gyration (13.2m) of the building to maintain dimensional consistency with the sway mode shapes. In other words, this technique allows for relative comparison of the mode shapes in the three principle directions. The first mode displays strong coupling action in the X and θ directions. Similarly, the building exhibits coupled action in the Y and θ directions torsion for the second mode.

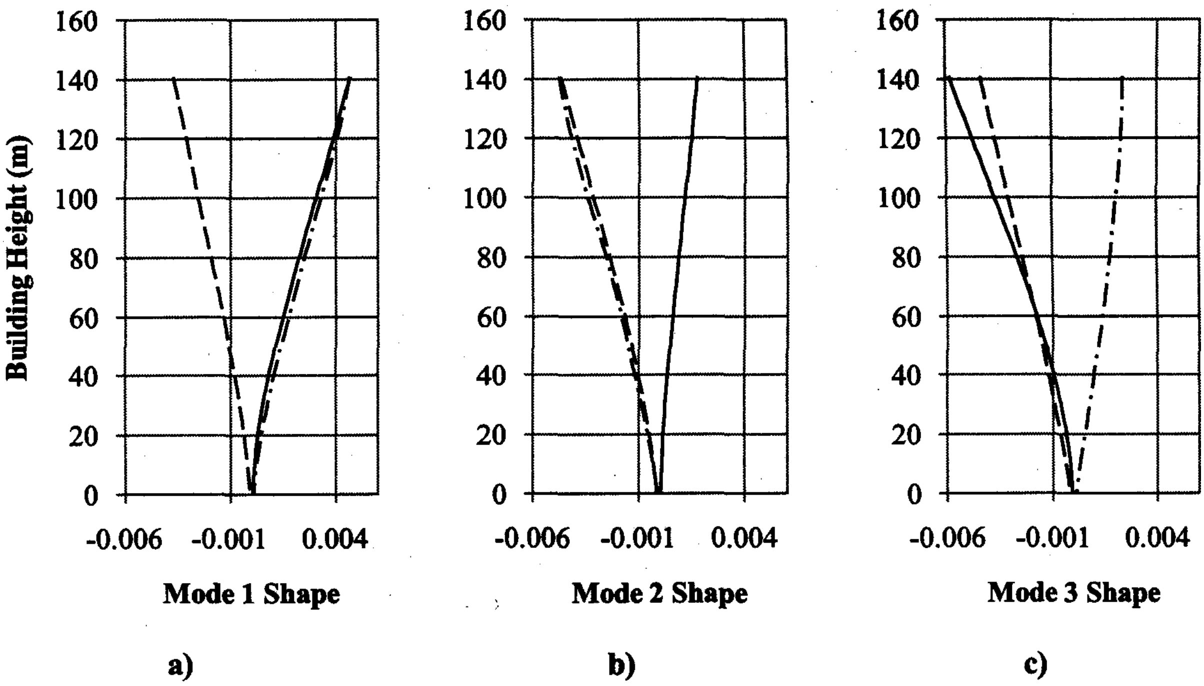


Figure 2.1 The mode shapes for sway in the X direction (solid line), sway in the Y direction (dashed line), and torsion (dash-dot line) for a) mode 1, b) mode 2, and c) mode 3.

The mode shapes are transformed from the center of masses (CM) to the center of coordinates (CC) as a means to align the building with the point where the HFFB measurements are recorded. In addition, this technique helps in linearizing the mode shapes (Tse et al., 2009) and assembling the lumped masses on a single vertical axis,

which simplifies the analysis of the building. The mode shapes at the CC are equal to

$$\begin{Bmatrix} \varphi_{xi} \\ \varphi_{yi} \\ \varphi_{\theta i} \end{Bmatrix}_{\text{coordinate}} = \begin{bmatrix} 1 & 0 & e_{yi} \\ 0 & 1 & -e_{xi} \\ 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} \varphi_{xi} \\ \varphi_{yi} \\ \varphi_{\theta i} \end{Bmatrix}_{\text{mass}} \tag{2.1}$$

where the new mass matrix must satisfy

$$[m]_{\text{mass}} \{\varphi^2\}_{\text{mass}} = [m]_{\text{coordinate}} \{\varphi^2\}_{\text{coordinate}} \tag{2.2}$$

The generalized quantities used for analysis are not altered by the transformation of the reference axes from the CM to the CC (Yip, 1995). Figure 2.2 illustrates the location of the CC and the sign conventions used in the analysis. The generalized properties of the building, which are used in the analysis, are calculated using the mode shapes in Figure 2.1 and are summarized in Table 2.1. Using the exact value for the damping ratio is very important in the design stage of high-rise buildings; therefore, to reduce the uncertainty, the building is solved with two separate damping ratios – one and two percent.

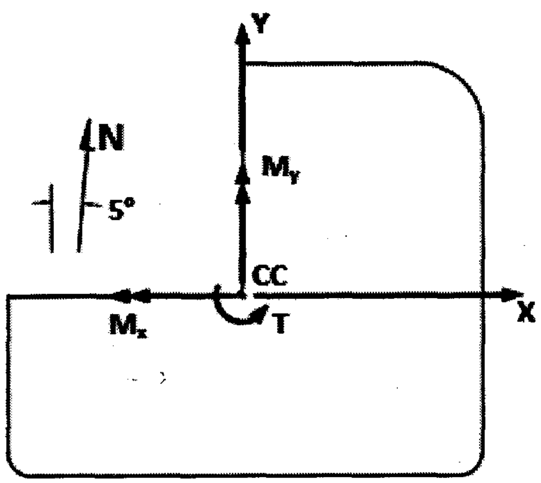


Figure 2.2 The building floor plan illustrating the center of coordinates and the sign conventions.

Table 2.1 The modal frequencies and generalized properties for the first three modes of the building.

Mode j	f_j (Hz)	m_j^* (kg)	k_j^* (N/m)	c_j^* (kg/s) (1%)	c_j^* (kg/s) (2%)
1	0.1588	988.8	984.0	19.7	39.5
2	0.1887	983.6	1383.2	23.3	46.7
3	0.2645	978.1	2700.9	32.5	65.0

2.3 Response of the Building to Wind Loading

The motions due to twisting moments can be particularly disturbing to occupants in a high-rise building. To check the comfort levels of the previously described torsionally sensitive building, wind tunnel testing is required to predict the response under design wind loads. The performance of the building is evaluated using the HFFB testing technique at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario. Rigid lightweight foam is used to produce a model of the building with a geometric length scale of 1:400. The model is mounted on a balance, which is capable of recording the base shear forces, bending moments, and torque. The HFFB testing technique sufficiently predicts the torsional response and accommodates 3D coupled mode shapes (Tschanz, 1982; Yip and Flay, 1995).

The main advantage of the HFFB technique is the ability to predict the generalized wind force (GWF), $F_j^*(t)$, directly from the measured base overturning and torsional moments. The GWF takes the form

$$F_j^*(t) = a_{xj}\frac{1}{H}C_{xj}M_x(t) + a_{yj}\frac{1}{H}C_{yj}M_y(t) + 0.7a_{\theta j}C_{\theta j}M_\theta(t) \tag{2.3}$$

where $M_\gamma(t)$ is the time-dependent base moment for direction γ (where $\gamma = x, y, \theta$) recorded from the HFFB test and scaled by the design wind speed and model length scale. In the formulation of the GWF, the assumption is made that the mode shapes are linear in

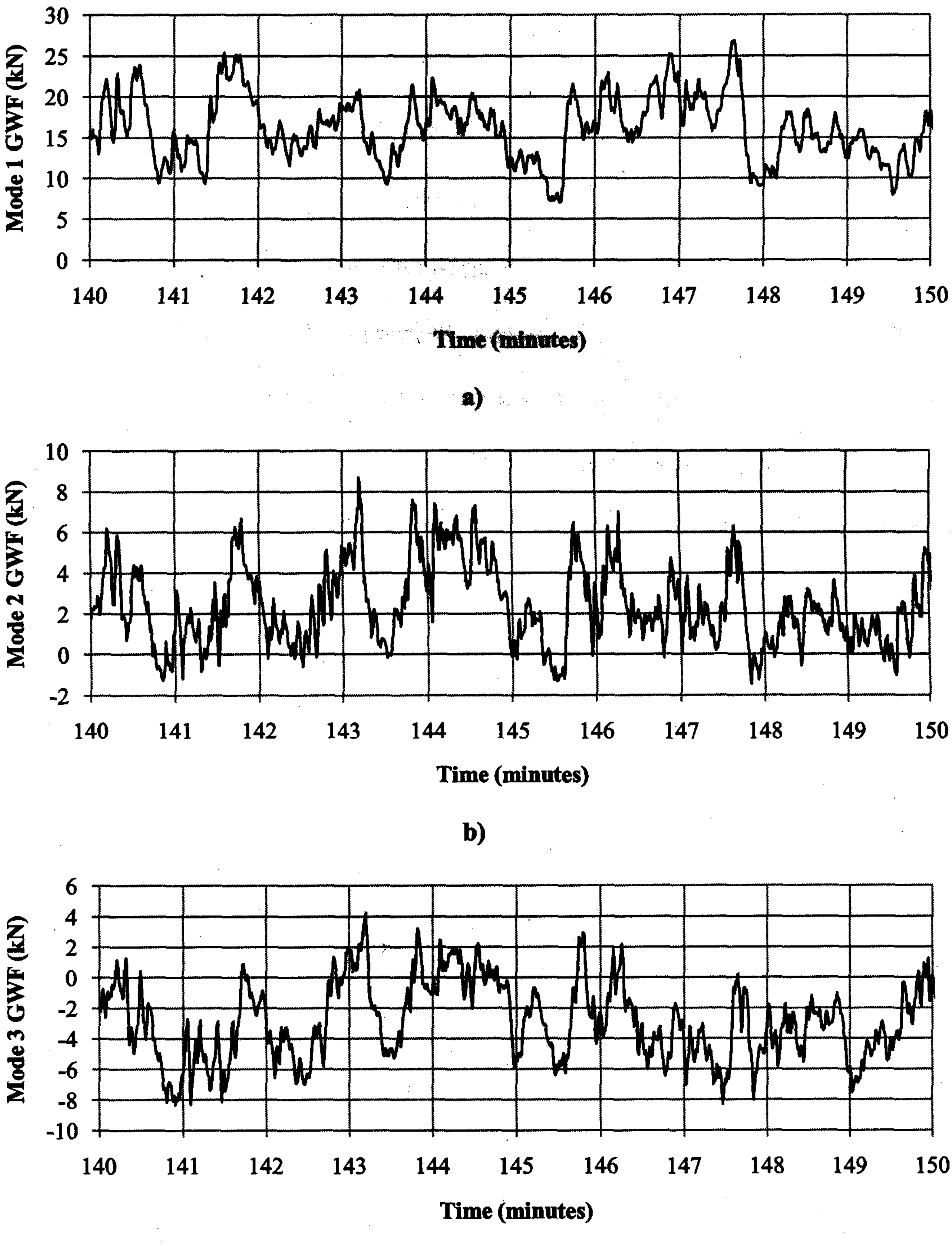
the sway directions and constant in the torsional direction; therefore, mode shape correction factors, $C_{\gamma j}$, are applied to the base moments to linearise the mode shapes for direction γ. Table 2.2 shows the correction factors, which depend on the power exponent of both the mode shape and the mean wind speed profile (Lam and Li, 2009). However, the mean wind speed profile causes minimal change in the correction factor, thus is generally ignored (Zhou et al., 1999b). Furthermore, the base torsional moment requires an empirical correction of 0.7 to transform the mode shape from linear to constant. For 3D coupled mode shapes, the relative contributions from each direction are accounted for by applying modal mixing factors, $a_{\gamma j}$ (Yip and Flay, 1995). The modal mixing factors are equal to the mode shape value at the CC of the top floor.

Table 2.2 The correction factors for the nonlinear mode shapes.

Mode j	C_{xj}	C_{yj}	$C_{\theta j}$
1	0.91	0.96	0.99
2	0.92	0.96	1.02
3	0.90	1.01	1.40

HFFB tests were conducted every 10 degrees of azimuth and at a few other critical wind directions with various wind exposures for a total of 43 tests. Surrounding buildings were modelled within a 550-meter radius and cubic roughness elements were raised upwind of the model. Each test lasted 250 seconds (approximately 4 hours in full-scale) and data were sampled at a rate of 116 hertz. The meteorological wind climate model is applied to this data to give the full-scale base shears and moments. The site-specific wind climate model for the building in this study predicts a mean hourly wind speed of 41.7m/s at 500 meters for a 10-year return period – the return period for serviceability responses. Using this model, the GWF for the first three structural modes is

calculated and a portion 10-minute portion is shown in Figure 2.3 for the 245 degrees of azimuth wind direction.



c)

Figure 2.3 A 10-minute segment of the generalized wind force at a 245° angle of incidence for a) mode 1, b) mode 2, and c) mode 3.

Classical modal analysis is used to solve the responses of the building subjected to the GWF. The structural modes are decoupled and solved independently by using the generalized properties of the building and GWF for mode j. Fundamentally, the building is transformed into an SDOF system described by

$$m_j^* \ddot{\xi}_{sj}(t) + c_j^* \dot{\xi}_{sj}(t) + k_j^* \xi_{sj}(t) = F_j^*(t) \tag{2.4}$$

where $\ddot{\xi}_{sj}$, $\dot{\xi}_{sj}$, and ξ_{sj} are the generalized acceleration, velocity, and displacement responses of the structure. The equation of motion is solved in the time domain using the fourth-order Runge-Kutta-Gill numerical method to obtain the modal generalized responses, which are transformed to the physical modal responses by use of the mode shapes. The modal responses are combined through summation. From the time series for each response, the Lieblein BLUE technique is employed for the determination of the statistical parameters of a Type I extreme-value distribution (Lieblein, 1974). The technique provides a probabilistic model, thus resulting in a more controlled and accurate prediction of the peak responses of the building. Additionally, the probability distribution accommodates the calculation of 1-hour peak responses for a given confidence interval.

The 43 tests are solved in this manner and combined by response-specific importance factors, $\beta(\alpha)$. These factors describe the relative importance of each wind angle tested and are calculated from the site-specific wind climate model obtained from the BLWTL. The factors sum to unity. The result is a peak and root-mean-square (RMS) response variable for each serviceability limit issue as shown in Table 2.3 for a 1% and 2% damping ratio. The response variables are determined using a computer numerical model developed specifically for this study. The RMS response is a measure of the variance in the response variable and provides insight into the degree of fluctuation of the response.

Table 2.3 The peak and RMS serviceability responses of the building.

Response Variable	1% Damping		2% Damping	
	Peak	RMS	Peak	RMS
X Acceleration (milli-g)	17.1	7.4	12.2	5.2
Y Acceleration (milli-g)	17.1	7.4	12.2	5.2
Torsional Acceleration (milli-g)	31.6	13.4	21.6	9.2
Centroidal Acceleration (milli-g)	19.8	5.1	14.3	3.6
Corner Acceleration (milli-g)	35.4	9.6	25.5	6.8
Torsional Velocity (milli-rad/s)	11.5	5.3	8.2	3.6
X Deflection (mm)	288	82	258	65
Y Deflection (mm)	277	84	252	69
Torsional Deflection (mm)	481	146	411	110

The accelerations are calculated at the uppermost occupied floor ($z = 140.7$m) and are expressed in terms of the gravitational acceleration (milli-g). The torsional acceleration is calculated using a lever arm of 23.0 meters – the furthest occupiable distance from the CC. The centroidal acceleration is the maximum combined acceleration at the CC. The corner acceleration is the absolute maximum acceleration experienced from the collection of the five corners of the floor plan. The deflections are calculated at the top of the building ($z = 161.7$ meters) and the lever arm for the torsional deflection is 24.0 meters.

The magnitudes of the peak torsional responses are much greater than the peak lateral responses, which clearly demonstrate the torsional irregularity of the building. The torsion sensitivity exacerbates the high translational accelerations by drastically increasing them as the distance away from the CC increases, thereby causing the combined accelerations to exceed recommended limits. Criteria for acceptable wind-induced motions are related to human perception thresholds, which are calculated using a probabilistic approach and experimental evaluation. Based on this concept, the BLWTL has recommended the following criteria for acceptable accelerations: 10 to 15 milli-g for residential buildings, 15 to 20 milli-g for hotels, and 20 to 25 milli-g for office buildings

(Isyumov, 1994). In addition to the excessive building accelerations, the torsional velocity exceeds the recommended maximum of 5 milli-rad/s. The serviceability criteria in the National Building Code of Canada (NBCC) only addresses inter-storey drift; therefore, the serviceability criteria dictated by the BLWTL are taken as the acceptable limits.

2.4 Description of Tuned Liquid Dampers

To ensure the building responses remain below the perception threshold, the use of a DVA is necessary to increase the damping. Tuned liquid dampers are chosen because they reach peak efficiency in the hourly peak acceleration range of 12 to 22 milli-g, corresponding to a once in 10-year exceedance (Tait et al., 2008b). Therefore, a properly designed TLD system is an optimal DVA for the building in this study.

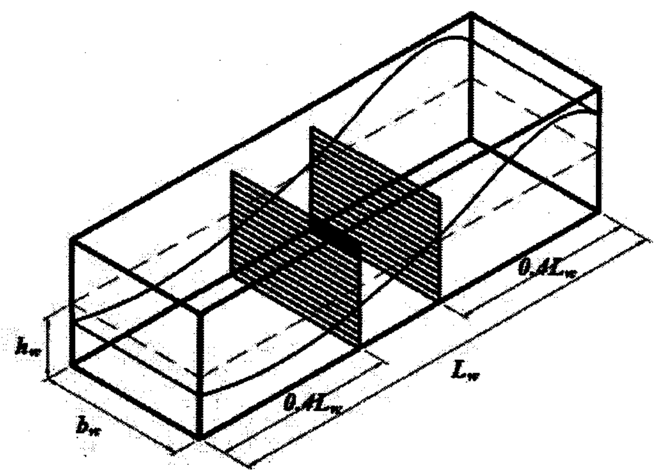


Figure 2.4 A diagram of a typical rectangular TLD showing the dimensions and the screen locations.

The tuning ratio and mass ratio are important parameters that strongly influence the performance of the TLD. They both depend on the tank length, L_w, width, b_w , and water height, h_w , shown in Figure 2.4 along with the placement of the damping screens. The tuning ratio, Ω, is defined as the ratio of the sloshing frequency to the structural frequency

in the mode that the dynamic motions are to be suppressed. The mass ratio, μ, is defined as the ratio of the generalized participating water mass to the structural generalized mass. The ratios take the form

$$\Omega = \frac{f_w}{f_s} \tag{2.5}$$

$$\mu = \frac{\alpha_p M_p}{m^*} \tag{2.6}$$

where α_p is the proportion of water mass that contributes to the fundamental sloshing mode of the TLD defined by equation 2.8 and m^* is the generalized mass of the building for the desired mode. M_p is the total generalized water mass defined by equations 2.9 and 2.10. Despite the amplitude-dependent nonlinear behaviour of a TLD, the use of linear wave theory to estimate the water sloshing frequency, f_w, and potential flow theory to estimate the water mass contributing to the fundamental sloshing mode, m_1, are acceptable for the initial design (Tait et al., 2005a). These estimations take the form

$$f_w = \frac{1}{2\pi}\sqrt{\frac{\pi g}{L_w}\tanh\left(\frac{\pi h_w}{L_w}\right)} \tag{2.7}$$

$$\alpha_p = \frac{m_1}{m_w} = \frac{8\tanh\left(\frac{\pi h_w}{L_w}\right)}{\pi^3\left(\frac{h_w}{L_w}\right)} \tag{2.8}$$

where g is the acceleration of gravity and m_w is the total mass of water in a single tank. An optimal TLD design is promoted by setting the tuning ratio to near unity and mass ratio to within a range of one to four percent (Tait, 2008a).

To characterize the nonlinear properties of a TLD, an equivalent amplitude-dependent tuned mass damper (TMD) approach is used (Sun et al., 1995). The concept involves matching the energy dissipation of the sloshing water to the energy dissipation of an

equivalent TMD for specific excitation amplitudes. The result is the TLD described by a frequency, f_{TLD}, mass, m_{TLD}, and damping ratio, ζ_{TLD}, which are dependent on the amplitude. Depending on the tank dimensions, these parameters are expressed as linear or power functions, which increase as the amplitude increases (Tait et al., 2004a).

2.4.1 Unique Designs for Coupled Motion

Three TLD systems are designed specifically for coupled lateral-torsional motion and are shown in Figure 2.5. They are denoted as TS-1, TS-2, and TS-3. The main difference between the three systems is the TLD mass ratio, which ranges from 1.28 to 4.94 percent. This is achieved by increasing the size and number of tanks. TS-1 uses 1D tanks, TS-2 utilises 2D tanks, and TS-3 employs two layers of 1D tanks. Tamura et al., (1996) demonstrated the effective use of floor space for multiple layers of TLDs in airport control towers, located in Japan. The 2D tanks are designed as two independent 1D tanks because the wave motions and base shear forces in the tanks are uncoupled in the two principle orthogonal directions (Tait et al., 2007).

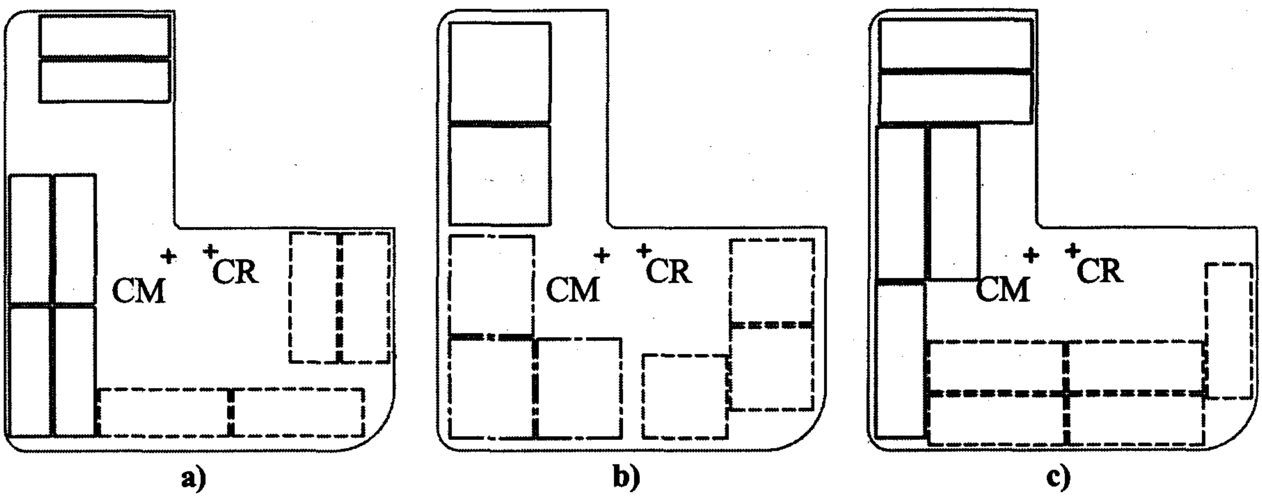


Figure 2.5 The layout of the TLDs tuned to mode 1 (solid line), mode 2 (dashed line), and both mode 1 and 2 (dash-dot line) for a) TS-1, b) TS-2, and c) TS-3.

The three TLD systems are designed for installation on the roof (z = 140.7 meters) of the building. Figure 2.1 illustrates that the first two modes have a high torsional contribution to the dynamic response. To improve the effectiveness of the vibration control, the first several modes should be controlled by TLDs (Li et al., 2004); therefore, each of the three TLD systems will have two sets of tanks – one set tuned to mode 1 and a second set tuned to mode 2. Note that TS-2 has a set of tanks tuned to both mode 1 and mode 2. To maximize the amplitude experienced by the TLD, the tanks are placed around the perimeter of the floor plan. This also increases the generalized water mass by introducing an eccentricity, L_x or L_y, between the tanks and the center of mass of the building. The generalized water mass for a single tank aligned in the X or Y direction, respectively, is calculated by

$$m_p = m_w\left(\varphi_x - L_y\varphi_\theta\right)^2 \quad \textbf{(2.9)}$$

$$m_p = m_w\left(\varphi_y + L_x\varphi_\theta\right)^2 \quad \textbf{(2.10)}$$

where φ_γ is the mode shape for direction γ. The summation of the generalized water mass, M_p, of the tanks for each system is shown in Table 2.4 for mode 1 and mode 2. Note that the ratio of the total water mass, M_w, to the structure mass, m_s, for TS-3 is similar to TS-2 but the TLD mass ratio, μ, is much greater for TS-3 because the floor plan space is used more effectively – the tanks of TS-3 have a greater eccentricity, L_x and L_y. The main difference between the designs is the generalized water mass, which consequently affects the TLD mass ratio, μ. The TLD system layouts are chosen to capture a wide range of TLD mass ratios such that TS-2 has a TLD mass ratio approximately twice that of TS-1 and two-thirds that of TS-3. This allows for investigation of the effect of the TLD mass ratio on the building response.

The sloshing frequency is directly related to the tank dimensions. The length and height of the tank are chosen to achieve a value of unity for the tuning ratio. Table 2.4 shows the dimensions chosen for the three systems along with the corresponding sloshing frequency. The height to length ratio is limited to a maximum of 0.2 to ensure the validity of the shallow water wave theory. Conversely, the ratio is kept sufficiently high to reduce the space requirements, the nonlinearity of the sloshing motion, and the potential wave breaking action inside the tank (Tait et al., 2005a). Moreover, the nonlinear response characteristics increase as the water depth to tank length ratio is reduced (Tait et al., 2004a).

Table 2.4 The tank dimensions, sloshing frequency, mass characteristics, and optimal effective damping for each mode of the three TLD systems.

	Mode	L_w (m)	h_w (m)	f_w (Hz)	α_p	M_p (kg)	M_w/m_s (%)	μ (%)	$\zeta_{eff-opt}$
TS-1	1	10.4	1.157	0.1588	0.779	16.3	0.37	1.28	0.0283
	2	10.4	1.708	0.1887	0.746	19.3	0.42	1.46	0.0303
TS-2	1	8.0	0.673	0.1588	0.792	37.2	1.15	2.98	0.0433
	2	6.7	0.673	0.1887	0.785	45.8	1.14	3.65	0.0480
TS-3	1	12.4	1.674	0.1588	0.765	54.6	1.30	4.22	0.0516
	2	10.8	1.855	0.1887	0.740	65.7	1.25	4.94	0.0559

The use of damping screens in the tanks is important to increase the inherent damping of the TLD. Without screens, the inherent damping relies on viscous dissipation in the boundary layers at the walls and bottom of the tank and from free surface contamination. This results in an inherent damping that is significantly less than optimal (Fediw et al., 1995). To achieve the effective optimal damping ratio shown in Table 2.4, this study uses slat screens in all the tanks. Two screens, with a solidity ratio of 0.42, placed at the $0.4L_w$ and $0.6L_w$ locations provide a near optimal inherent damping. The utilization of

damping screens also reduces the nonlinearity of the TLD by removing the higher sloshing harmonics, and reducing the maximum wave height, thus reducing the chance of wave breaking for any given amplitude of excitation (Tait et al., 2005a). Wave breaking will occur approximately when the water at the end walls reaches a height of $2h_w$ (Sun and Fujino, 1994).

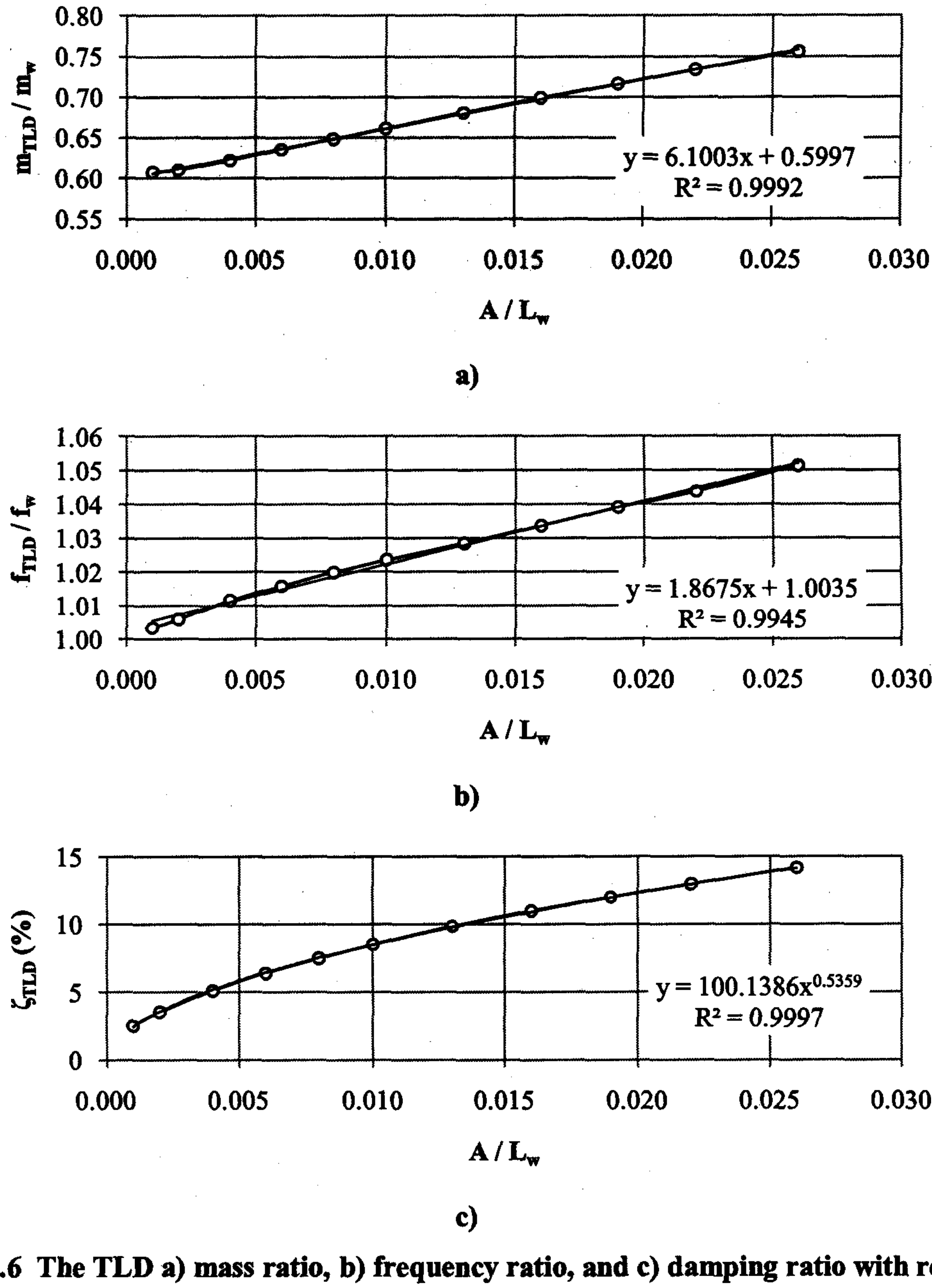


Figure 2.6 The TLD a) mass ratio, b) frequency ratio, and c) damping ratio with respect to the normalized amplitude of excitation for the mode 1 tanks of TS-3.

Using a numerical model developed by Tait et al. (2004a), a set of amplitude-dependent TLD properties are evaluated. The program performs discrete frequency sweep tests on the TLD at a desired amplitude of excitation, A, to produce an equivalent TMD frequency, mass, and damping ratio. The three amplitude-dependent properties for mode 1 tanks of TS-3 are shown in Figure 2.6 with a fitted curve and the corresponding R^2 value. The R^2 value describes how well the curve fits the data points (unity is a perfect fit). A unique set of these properties are determined for each mode of the three TLD systems and are used in the response evaluation of the structure-TLD system.

2.5 Response of the Building with a TLD System

2.5.1 Structure-TLD Numerical Model

Tait et al. (2004b) formulated a method to analyse the complex structure-TLD system with the TLD tuned to a particular mode. Figure 2.7b illustrates the building characterized by a generalized mass, m_j^*, stiffness, k_j^*, and damping, c_j^*, for mode j. The TLD interacts with the generalized building through a second DOF with amplitude-dependent equivalent TMD properties as shown in Figure 2.7c. The equation of motion for the 2DOF system takes the form

$$\begin{aligned} &\begin{bmatrix} m_j^* + m_o & 0 \\ 0 & m_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \ddot{\xi}_s(t) \\ \ddot{\xi}_{TLD}(t) \end{Bmatrix} + \begin{bmatrix} c_j^* + c_{TLD}(A) & -c_{TLD}(A) \\ -c_{TLD}(A) & c_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \dot{\xi}_s(t) \\ \dot{\xi}_{TLD}(t) \end{Bmatrix} \\ &+ \begin{bmatrix} k_j^* + k_{TLD}(A) & -k_{TLD}(A) \\ -k_{TLD}(A) & k_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \xi_s(t) \\ \xi_{TLD}(t) \end{Bmatrix} = \begin{Bmatrix} F_j^*(t) \\ 0 \end{Bmatrix} \end{aligned} \quad \textbf{(2.11)}$$

where m_{TLD}, $k_{TLD}, and\ c_{TLD}$, are the equivalent TMD mass, stiffness, and damping coefficient with respect to the amplitude, A. The excitation force is the GWF, $F_j^*(t)$. To account for the nonlinear dynamic properties of the TLD, the maximum displacement (or

amplitude) is calculated for cycle n and is used to update the TLD mass, m_{TLD}, damping coefficient, c_{TLD}, and stiffness, k_{TLD}, for cycle $n+1$ using the appropriate TLD equations – similar to the equations shown in Figure 2.6. However, before calculating the updated TLD properties, an amplitude modification factor (AMF), Φ_j, is applied to the maximum displacement to transform the generalized response to a physical response. This accounts for the combined lateral-torsional motion. The AMF is calculated using a normalized weighted-average of the water mass multiplied by the mode shape value at the tank location for each TLD. Table 2.5 summarizes the values used in the analysis. In addition, section 2.5.3.2 further discusses the role of the AMF. The analysis solves the structure-TLD system for the first two modes of vibration and combines them through addition with the third mode solved as an SDOF system because there is no TLD tuned to the third mode.

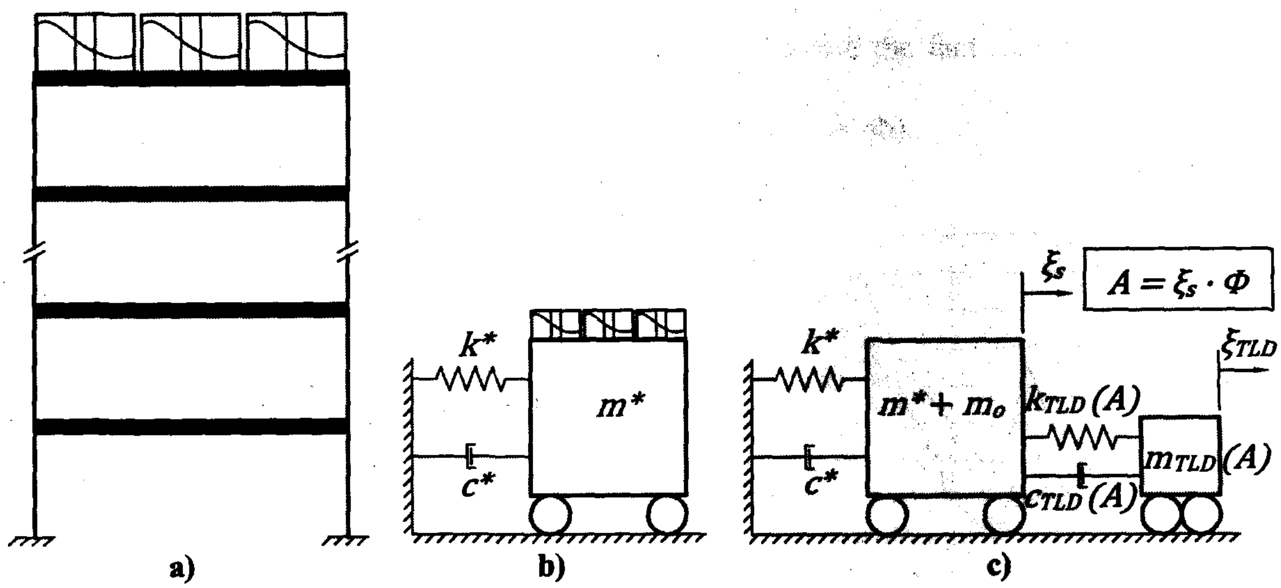


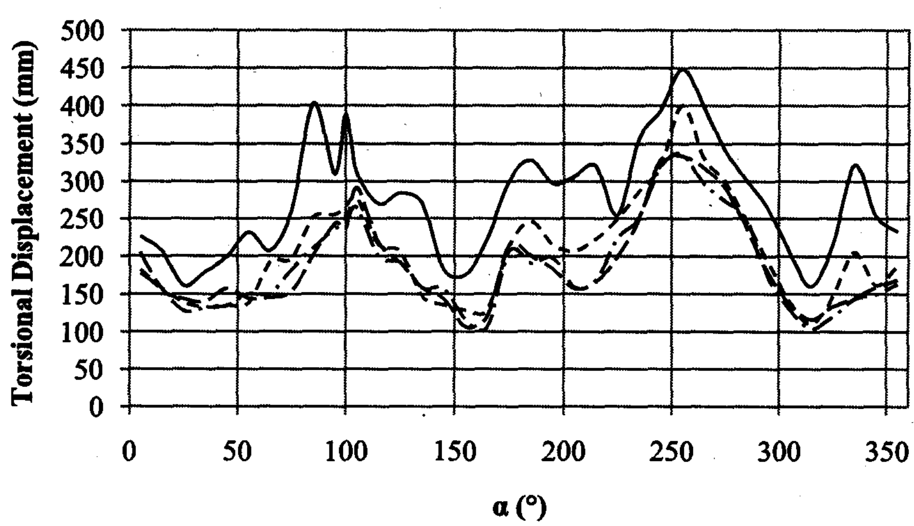
Figure 2.7 The transformation of a) the structure-TLD system into b) a generalized structural system with TLDs then into c) a 2DOF system.

Table 2.5 The amplitude modification factors for each mode of the three TLD systems.

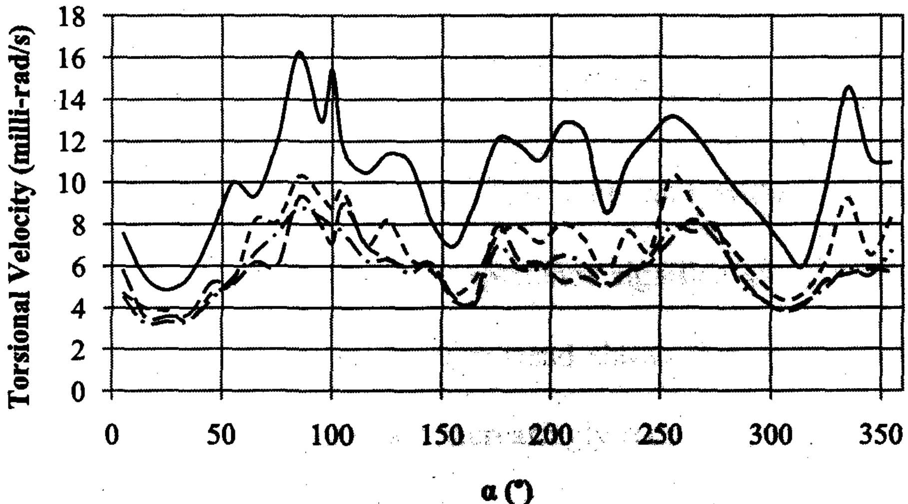
	Mode j	Φ_j
TS-1	1	0.00835
	2	0.00895
TS-2	1	0.00739
	2	0.00839
TS-3	1	0.00821
	2	0.00923

2.5.2 Performance of the Building

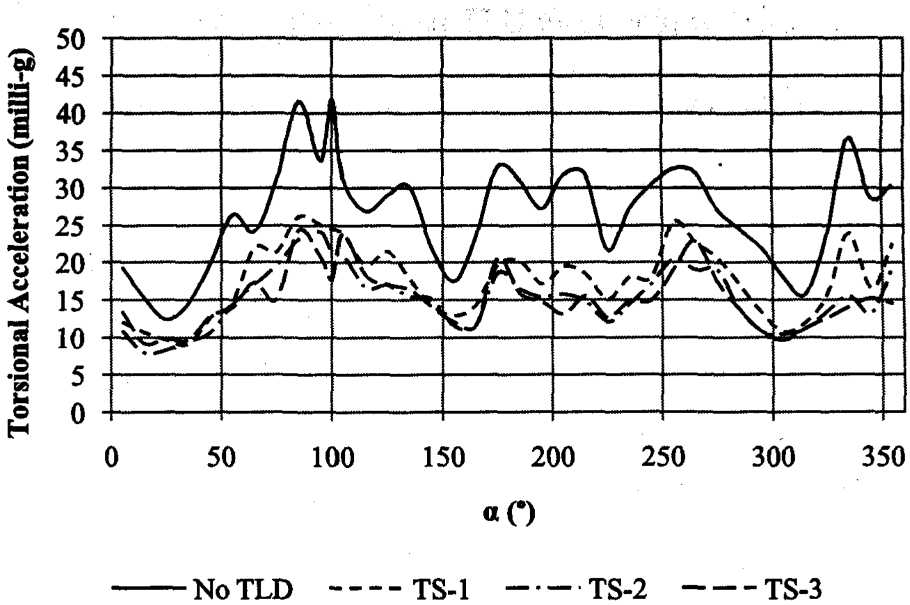
The peak and RMS building responses are solved for each of the 43 wind test angles, α, in degrees of the azimuth. Figure 2.8a)-c) shows the peak hourly torsional responses of the 1% damped building without a TLD system installed and with each of the three systems installed. All three TLD systems reduce the peak responses of the building; however, there is only a weak disparity between the effectiveness of each TLD system. At the wind angles associated with the peaks, the TLD systems offer a greater reduction. This demonstrates the nonlinearity of the TLD systems and the fact that a TLD is more effective when subjected to stronger motions (Tait et al., 2004b).



a)



b)



c)

Figure 2.8 The torsional a) displacement, b) velocity, and c) acceleration for the building with and without the TLD systems.

For the peak hourly responses of the building, irrespective of the wind direction, the site-specific wind climate model is applied to the responses for each wind direction. The probabilistic model accounts for the worst case wind direction through response-specific importance factors, $\beta(\alpha)$. Table 2.6 and Table 2.7 show the peak hourly responses for the building with a 1% and 2% damping ratio, respectively. Both the peak and RMS responses are calculated for each TLD system along with the corresponding percent

reduction, Ψ, calculated by

$$\Psi = \frac{R_{No\,TLD} - R_{With\,TLD}}{R_{No\,TLD}} \cdot 100 \tag{2.12}$$

where $R_{With\,TLD}$ and $R_{No\,TLD}$ are the responses with and without a TLD installed, respectively. The building occupants will experience significantly less motion with any one of the three TLD systems installed. The trend shows that as the TLD water mass ratio increases, the torsional responses are increasingly reduced, thus affirming that TS-3 is the most effective. However, TS-2 offers the greatest reduction in the translational responses. This occurs because the optimal TLD mass ratio is dependent on the response that is to have the greatest reduction. In other words, the maximum response reduction in the lateral direction is achieved at a lower TLD mass ratio than in the torsional direction. Furthermore, there are minimal differences in reductions between TS-2 and TS-3, thereby indicating that there is an upper limit on the optimal water mass ratio.

The TLD systems offer the greatest reduction for the torsional responses because they are specifically designed to damp the torsion motion. The implementation of TS-3 will reduce the accelerations to an acceptable level (less than 20 milli-g) except for the corner acceleration for the 1% damping case. However, the corner acceleration easily meets the recommended limit for an office building. None of the three TLD systems is capable of reducing the torsional velocity below the 5 milli-rad/s recommendation. However, the criteria for serviceability issues are conservative (Hansen et al., 1973). Therefore, the majority of the building occupants will not perceive the 6.5 milli-rad/s torsional velocity.

The responses for the 2% damped building show a further reduction from the 1% damping level; however, the torsional velocity is still slightly above the recommended limit. In general, the TLD systems are less effective when the building has a higher

damping ratio because the TLD effective damping is negatively dependent on the structural damping (Tait et al., 2004b). Nonetheless, a well-designed TLD system is an effective DVA for multi-modal coupled lateral-torsional motion.

Table 2.6 The peak and RMS response variables for the 1% damped building with each of the three different TLD systems installed.

Response Variable	TS-1		TS-2		TS-3	
	Peak	RMS	Peak	RMS	Peak	RMS
X Acceleration (milli-g)	13.9	5.8	13.0	5.5	12.9	5.5
	(19.1)	(21.3)	(23.9)	(26.2)	(25.0)	(26.7)
Y Acceleration (milli-g)	12.5	5.2	11.6	4.9	12.1	5.0
	(26.6)	(29.1)	(32.1)	(34.2)	(29.5)	(31.8)
Torsional Acceleration (milli-g)	20.5	8.5	17.7	7.3	17.5	7.3
	(35.2)	(36.5)	(44.0)	(45.7)	(44.7)	(45.7)
Centroidal Acceleration (milli-g)	14.8	3.8	13.7	3.5	13.7	3.6
	(25.2)	(26.6)	(30.7)	(31.1)	(30.9)	(30.5)
Corner Acceleration (milli-g)	24.8	6.3	21.8	5.4	20.6	5.1
	(30.1)	(34.1)	(38.6)	(43.6)	(41.9)	(46.6)
Torsional Velocity (milli-rad/s)	7.5	3.3	6.5	2.9	6.5	2.9
	(34.6)	(36.3)	(42.9)	(45.5)	(43.5)	(44.9)
X Deflection (mm)	258	65	253	61	255	61
	(10.7)	(20.8)	(12.2)	(25.3)	(11.6)	(25.2)
Y Deflection (mm)	246	67	241	65	242	66
	(11.1)	(20.0)	(13.1)	(23.3)	(12.8)	(21.3)
Torsional Deflection (mm)	399	104	380	95	381	96
	(17.0)	(28.4)	(20.9)	(35.0)	(20.7)	(34.1)

The RMS values are included to demonstrate the ability of the TLD systems to reduce the fluctuations of the building motion. The TLD systems are capable of reducing the RMS values more significantly than the peak responses. The TLD system essentially reduces the cyclical swaying and twisting of the building to levels that are significantly less perceptible than without the TLD system. There is still debate between whether the peak or RMS response is the best measure for serviceability limit criteria (Pagnini and

Solari, 1998). Regardless, a well-designed TLD system will significantly reduce the motion of a high-rise building.

Table 2.7 The peak and RMS response variables for the 2% damped building with each of the three different TLD systems installed.

Response Variable	TS-1		TS-2		TS-3	
	Peak	RMS	Peak	RMS	Peak	RMS
X Acceleration (milli-g)	10.8	4.5	10.3	4.2	10.2	4.2
	(11.5)	(14.1)	(15.7)	(19.0)	(16.6)	(19.6)
Y Acceleration (milli-g)	10.0	4.2	9.5	3.9	9.7	4.0
	(18.2)	(19.2)	(22.3)	(24.1)	(21.0)	(22.0)
Torsional Acceleration (milli-g)	16.4	6.8	14.3	5.9	14.1	5.8
	(24)	(25.8)	(33.7)	(35.6)	(34.6)	(36.5)
Centroidal Acceleration (milli-g)	12.1	3.0	11.2	2.8	11.1	2.8
	(15.8)	(17.8)	(21.7)	(22.8)	(22.2)	(22.3)
Corner Acceleration (milli-g)	20.2	5.2	18.1	4.5	17.2	4.3
	(20.8)	(23.2)	(28.9)	(32.9)	(32.4)	(36.3)
Torsional Velocity (milli-rad/s)	6.2	2.7	5.4	2.4	5.3	2.4
	(24.4)	(25.0)	(33.4)	(34.2)	(35.2)	(34.2)
X Deflection (mm)	247	59	245	57	244	57
	(4.1)	(10.1)	(4.8)	(13.3)	(5.3)	(13.1)
Y Deflection (mm)	239	63	235	61	236	62
	(5.3)	(9.2)	(6.9)	(11.3)	(6.5)	(9.9)
Torsional Deflection (mm)	376	92	363	86	363	87
	(8.4)	(16.1)	(11.5)	(21.5)	(11.7)	(20.9)

2.5.3 Parametric Study

To assess the robustness of the TLD system, a parametric study is performed. TS-3 displays the performance of interest by maximizing the reduction in the torsional response of the building; therefore, the parametric study is performed on this system. Three parameters are chosen for the study: the water height inside the tank, the amplitude modification factor, and the first and second mode structural frequencies of the building. These parameters are chosen because they are the most practical variables that affect the structure-TLD performance.

2.5.3.1 Effect of the Water Height

The water height inside the TLD is of upmost interest because it affects both the tuning ratio and mass ratio, which are the critical parameters of a well-designed TLD. Evaporation, spillage, leakage, and poor maintenance will cause the water height to fluctuate. The varying water height will simultaneously change the sloshing frequency and water mass. Ten numerical tests are conducted, each with a different water height, h, ranging between $\pm 25\%$ of the optimal water height, h_w. The water height is considered optimal when the tuning ratio is unity. The water height, frequency, and mass used in the ten tests are shown in Table 2.8 for mode 1 and mode 2.

Table 2.8 The tests performed to analyse the effect of the water height on the structure-TLD response.

Test ID	Mode 1					Mode 2				
	h (m)	f_w (Hz)	Ω	M_p (kg)	μ (%)	h (m)	f_w (Hz)	Ω	M_p (kg)	μ (%)
WH+25	2.09	0.175	1.10	68.2	5.12	2.32	0.206	1.09	82.1	5.90
WH+20	2.01	0.172	1.08	65.5	4.95	2.23	0.203	1.08	78.8	5.72
WH+15	1.93	0.169	1.06	62.8	4.77	2.13	0.200	1.06	75.5	5.53
WH+10	1.84	0.166	1.04	60.0	4.59	2.04	0.196	1.04	72.2	5.34
WH+5	1.76	0.162	1.02	57.3	4.41	1.95	0.193	1.02	68.9	5.14
WH0	1.67	0.159	1.00	54.6	4.22	1.86	0.189	1.00	65.7	4.94
WH-5	1.59	0.155	0.98	51.8	4.03	1.76	0.185	0.98	62.4	4.73
WH-10	1.51	0.151	0.95	49.1	3.84	1.67	0.181	0.96	59.1	4.52
WH-15	1.42	0.148	0.93	46.4	3.65	1.58	0.176	0.93	55.8	4.30
WH-20	1.34	0.143	0.90	43.7	3.45	1.48	0.171	0.91	52.5	4.08
WH-25	1.26	0.139	0.88	40.9	3.25	1.39	0.167	0.88	49.2	3.85

Figure 2.9 shows the influence of the water height on the peak hourly responses. The results show that even a small change in the water height yields a significant change in the reduction of the peak responses. Moreover, the torsional responses, including the corner acceleration, are more sensitive than the lateral responses mainly because the

building exhibits a torsional sensitivity. However, this sensitivity to changes in the water height strictly enhances the performance of the TLD system because of the reasoning described in the following paragraph.

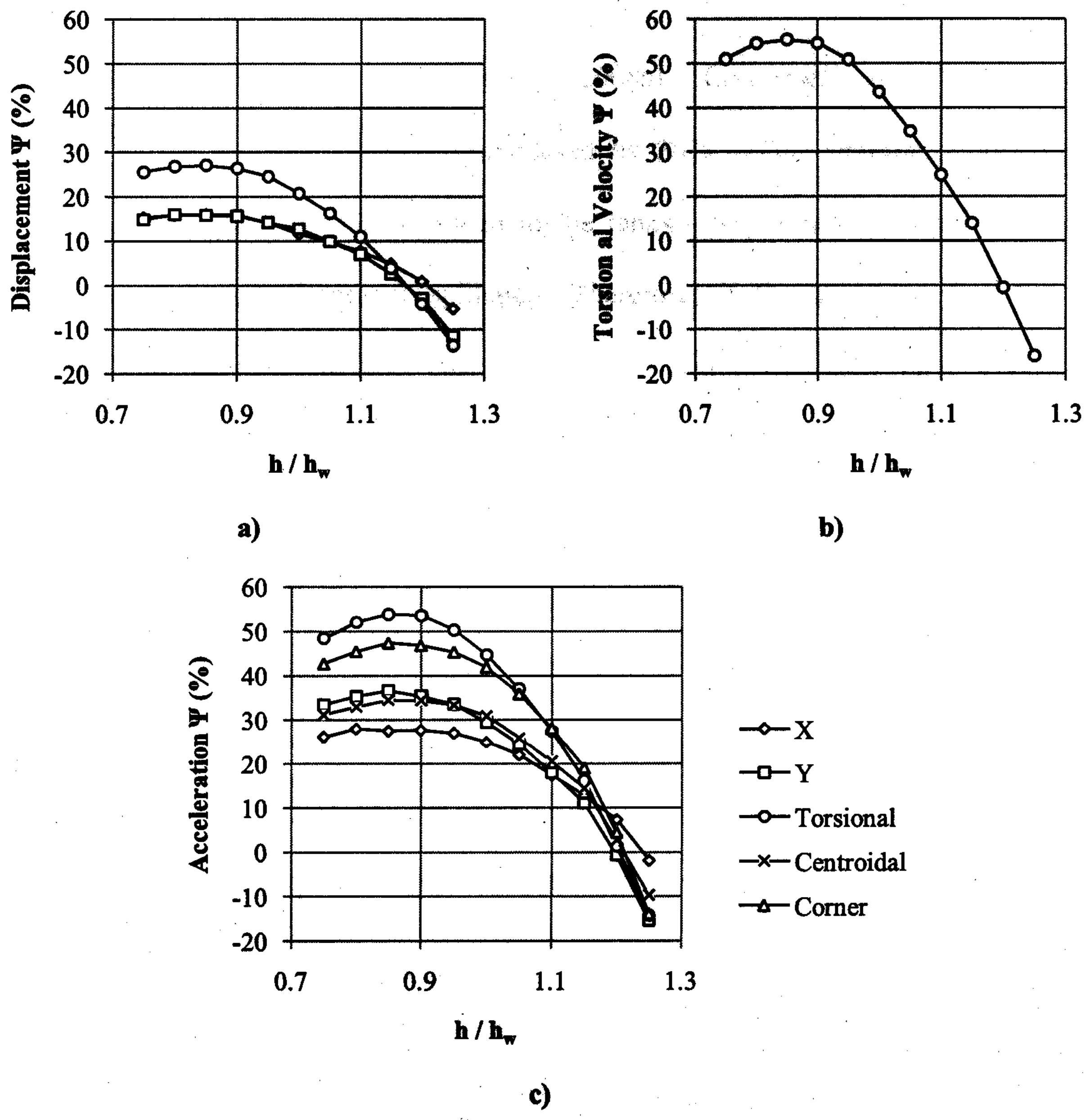


Figure 2.9 The effect of water height on the reduction in the a) displacement, b) torsional velocity, and c) acceleration responses of the building.

The results demonstrate that TS-3 is optimally effective at a reduced water height, approximately 15% less than the designed water height. A reduced water height will

lower the sloshing frequency, which increases the effectiveness of the TLD. The approximate optimal tuning ratio is given by

$$\Omega_{opt} = \frac{\sqrt{1+\frac{\mu}{2}}}{1+\mu} \tag{2.13}$$

where μ is the TLD water mass ratio (Tait, 2008a). Conversely, the TS-3 becomes ineffective when the water height is increased by 20%. This situation is extremely unlikely because water cannot be added to the tanks – water is more easily removed through evaporation, leakage, or spillage. Therefore, if the structural frequency is accurately calculated, a realistic change in water height will only have a positive influence on the structure-TLD system. Moreover, a substantial change in water height is unlikely to go unnoticed.

2.5.3.2 Effect of the Amplitude Modification Factor

The amplitude modification factor (AMF) is introduced in the numerical model to transform the generalized displacement to the real lateral-torsional coupled displacement experienced by the TLD system. For analysis purposes, the multiple tanks are essentially lumped into a single entity, thus removing the complexity of solving each tank separately. The AMF basically transforms the multiple tank system into a single tank with a generalized water mass equalling the summation of all the tanks. This is possible because the sloshing frequency, damping ratio, and TLD property equations remain constant between each tank. The AMF combines each tank into a single entity through a normalized weighted-average of the water mass multiplied by the mode shape at the TLD location for each TLD. The AMF is dependent on the distance, L_Y, between the CM of the building and the center of mass of each TLD. The distance, L_Y, has an upper limit of

the perimeter of the building and a lower limit of zero (at the CM). The amplitude modification factor, Φ, is numerically tested between these limits. The values are shown in Table 2.9. By varying the distance between the CM and the tanks, the total generalized water mass, M_p, changes. This value and the corresponding maximum displacement, X_{TLD}, used in the TLD equations to calculate the properties are also tabulated. The displacement is simply the maximum generalized displacement multiplied by the AMF.

Table 2.9 The tests conducted on the amplitude modification factor.

Test ID	Mode 1			Mode 2		
	Φ	M_p (kg)	A (mm)	Φ	M_p (kg)	A (mm)
AMF+2	0.00965	77.2	409	0.01066	88.9	147
AMF+1	0.00893	65.9	377	0.00995	77.3	133
AMF0	0.00821	54.6	344	0.00923	65.7	127
AMF-1	0.00742	46.7	301	0.00824	55.4	113
AMF-2	0.00663	38.9	260	0.00725	45.2	98
AMF-3	0.00584	31.1	231	0.00626	35.0	89
AMF-4	0.00505	23.2	212	0.00527	24.7	77
AMF-5	0.00426	15.4	192	0.00428	14.5	70

Figure 2.10 illustrates the effect of the AMF on the peak hourly response variables. The study reveals a substantial effect due to the change in eccentricity of the tanks. Noticeably, the torsion responses are more sensitive to these changes than the lateral responses. This demonstrates the effectiveness of using the eccentric mass technique to enhance the performance of the TLD system in reducing torsional motions. Furthermore, the lateral responses reach a maximum reduction (at the AMF-3 test) with a lower generalized water mass than for the torsional responses (at the AMF+1 test). This confirms the previously discussed limitation on the reduction of the lateral and torsional responses demonstrated between TS-2 and TS-3.

These numerical tests also introduce the influence of the TLD mass, frequency, and damping ratio on the response reduction because of the different amplitudes experienced at the CM compared to the perimeter of the floor plan (see Figure 2.6 at amplitudes equalling X_{TLD}). This study demonstrates that the AMF technique provides an acceptable approach for calculating TLD properties.

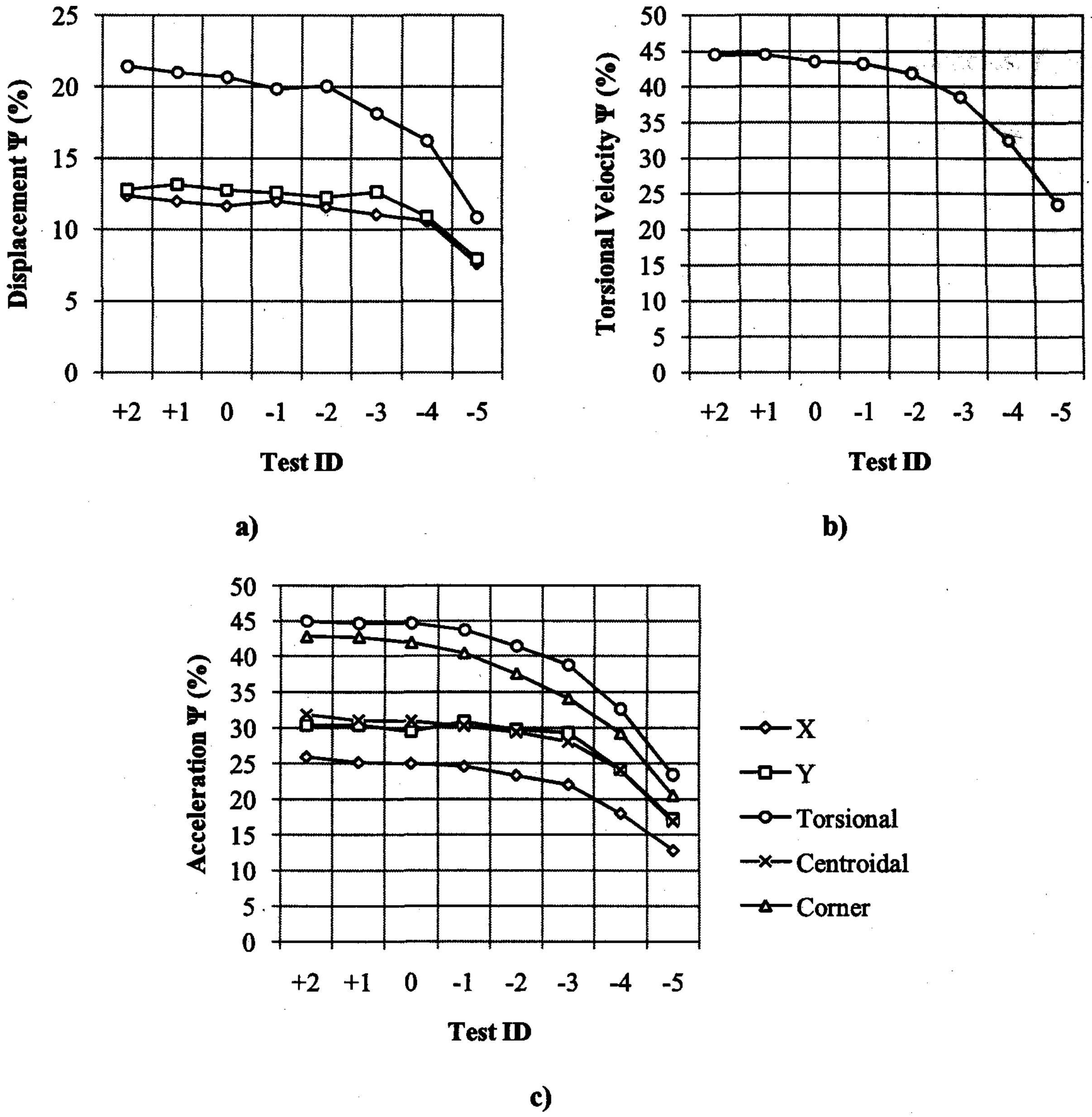


Figure 2.10 The effect of the AMF on the reduction in the a) displacement, b) torsional velocity, and c) acceleration responses of the building.

2.5.3.3 Effect of the Structural Frequencies

The most important parameter for TLD design is the tuning ratio. The TLD sloshing frequency is well defined; however, the possibility exists that the as-built structural frequencies will deviate from the estimated structural frequencies determined from a computer model by over 15% (Li et al., 2007). This leads to a mistuning of the TLD system. To investigate this issue, the structural frequencies, f_s, of the building for the first and second modes are varied separately by $\pm15\%$ of the modal frequencies, f_1 and f_2, obtained from the dynamic analysis. Table 2.10 summarizes the test parameters.

Table 2.10 The tests conducted to analyse the effect of the tuning ratios for mode 1 and mode 2 on the response reduction.

Test ID	Mode 1 Test		Mode 2 Test	
	f_s (Hz)	Ω	f_s (Hz)	Ω
FS+15	0.183	0.870	0.217	0.870
FS+12.5	0.179	0.889	0.212	0.889
FS+10	0.175	0.909	0.208	0.909
FS+7.5	0.171	0.930	0.203	0.930
FS +5	0.167	0.952	0.198	0.952
FS +2.5	0.163	0.976	0.194	0.976
FS0	0.159	1.000	0.189	1.000
FS-2.5	0.155	1.026	0.184	1.026
FS-5	0.151	1.053	0.179	1.053
FS-7.5	0.147	1.081	0.175	1.081
FS-10	0.143	1.111	0.170	1.111
FS-12.5	0.139	1.143	0.165	1.143
FS-15	0.135	1.176	0.160	1.176

Figure 2.11 shows the peak hourly responses of the building for the mode 1 test. The responses without a TLD system fluctuate because the frequency content of the wind excites the structure differently. The responses with a TLD system fluctuate less, thereby implying that the TLD system regulates the structural motion regardless of the frequency

content of the wind. In addition, the responses with a TLD system remain significantly less than the responses without a TLD system for underestimations of the structural frequency. In this range, the accelerations and torsional velocity remain below the responses without a TLD system because this tuning ratio range is near optimal (slightly less than unity) as described by equation 2.13. Conversely, the effectiveness of the TLD system rapidly decreases if the structural frequency is overestimated. Again, this is related to the tuning ratio departing from the optimal range. In fact, at a structural frequency overestimation of 15%, the TLD system begins to exacerbate the building responses. Notably, the displacements are more sensitive than the accelerations and torsional velocity.

The displacements are far less sensitive to changes in the second mode structural frequency as illustrated in Figure 2.12g)-i). In addition, the responses in the *X* direction are affected far less by the second mode structural frequency than the first mode structural frequency is that. This is because there is minimal dynamic contribution in the *X* direction for the second mode, as illustrated in Figure 2.1. Similar to the study on the first mode structural frequency, the *Y* and θ responses are affected by the tuning ratio. However, overestimations of the second mode structural frequency will not be as severe, as illustrated in Figure 2.12, because the effect of the mode 1 TLDs on mode 2 is not accounted for in this analysis. In other words, the mode 1 tanks will have a greater effect on the structure-TLD system for large underestimations of the second mode structural frequency; therefore, likely eliminating the ineffective portion (less than $f_s/f_2 = 1.0$) of the graphs in Figure 2.12.

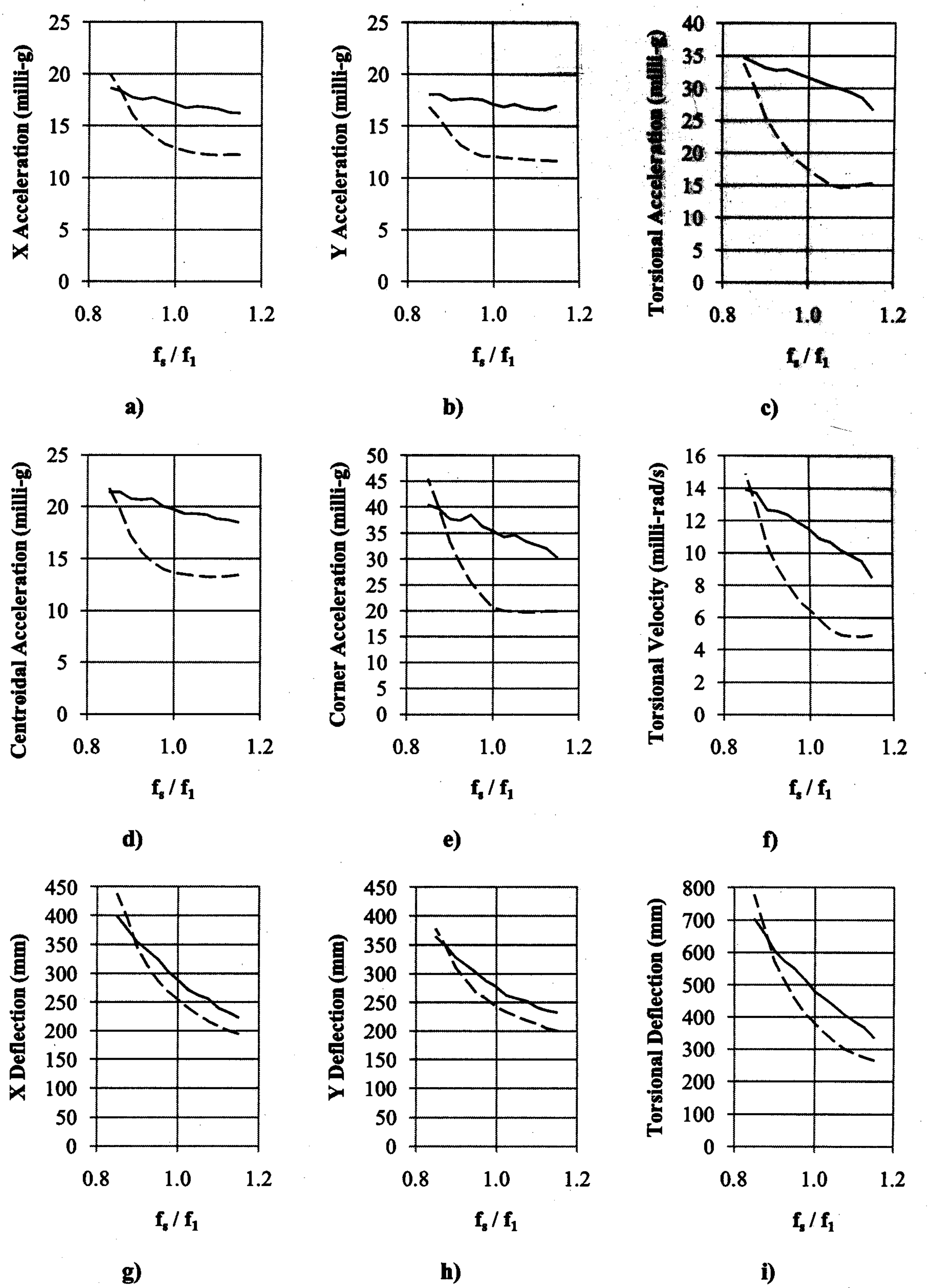


Figure 2.11 The peak responses of the building without a TLD system (solid line) and with a TLD system (dashed line) for different first mode structural frequencies.

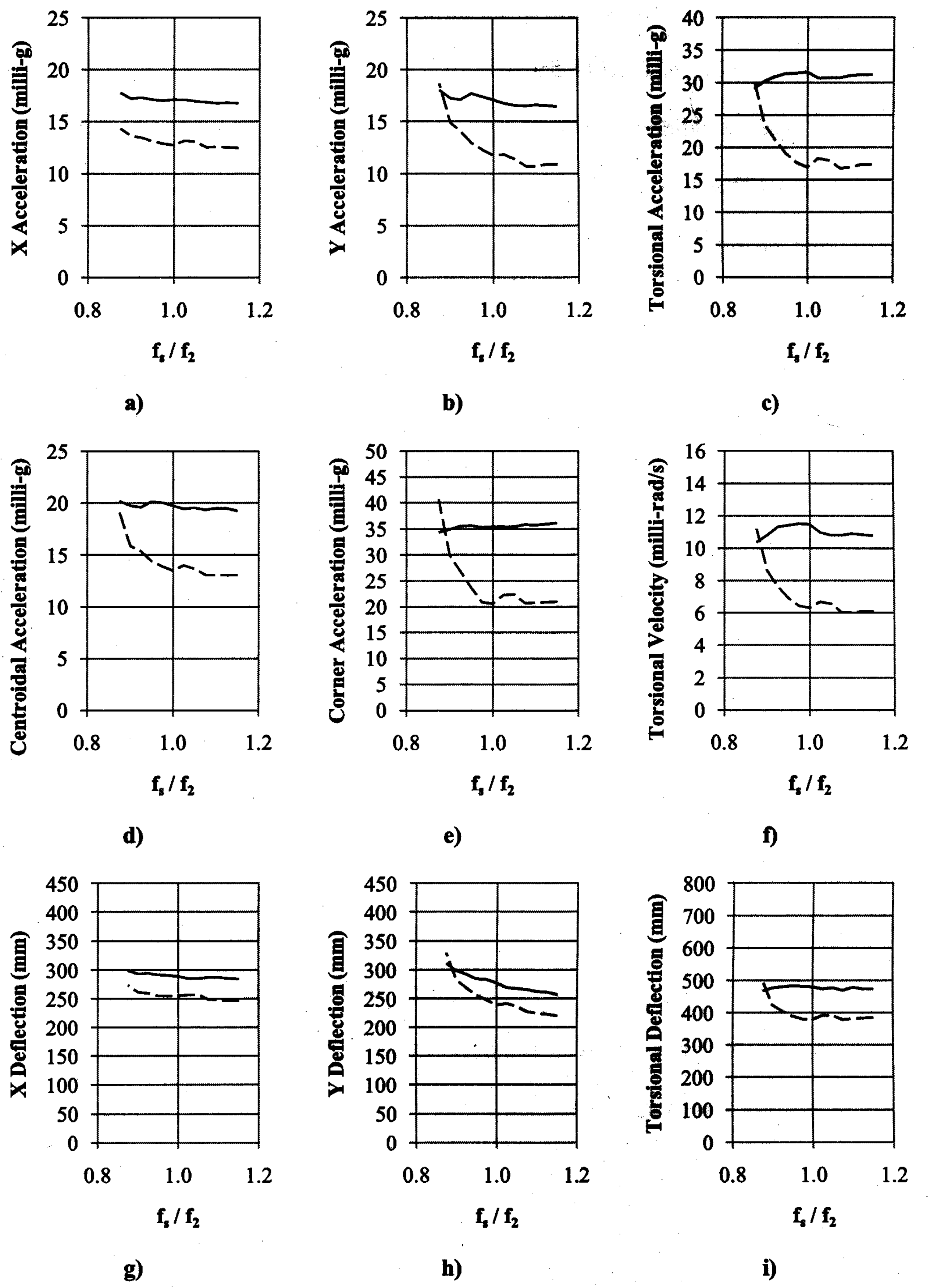


Figure 2.12 The peak responses of the building without a TLD system (solid line) and with a TLD system (dashed line) for different second mode structural frequencies.

The ability of the TLD system to regulate the structural motions, despite a mistuning, exemplifies the inherent robustness of a well-designed TLD. The TLD system is capable of adapting to a range of tuning ratios, thus increasing the flexibility of the TLD design. This flexibility accommodates fine-tuning after the installation of the TLDs by means of simply adding or removing water. A practical application is fine-tuning the TLDs to match the as-built structural frequencies, which can be determined using in-situ motion monitoring equipment in conjunction with the random decrement technique. A second application involves tuning the TLDs to a range of sloshing frequencies around the structural frequency of interest called a multiple tuned liquid damper (MTLD) system (Fujino and Sun, 1993). This 'mistuning' improves the effectiveness and robustness of the TLD system by increasing the adaptability to the fluctuating frequency content of the wind and the potentially variable structural frequencies.

2.6 Conclusion

In this chapter, three multi-modal TLD systems are described and tested on a lateral-torsional coupled building. The building in this study demonstrates coupled lateral-torsional action for the first three modes. Each TLD system is designed to suppress the first two modes by tuning two sets of tanks to the corresponding structural frequency of the building. In addition, the systems are designed to suppress the torsional motion by placing the tanks around the perimeter of the floor plan to maximize the inertial water mass from the coupled action.

The building with each of the three TLD systems is subjected to wind forces obtained from HFFB tests at the BLWTL at the University of Western Ontario. The results from solving the structure-TLD system demonstrates that the three TLD systems are capable of

significantly reducing the motions of the building, specifically the twist. As the water mass is increased, the ability for the TLD to suppress the torsional motion is increased; however, there is an upper limit to the TLD effectiveness with respect to water mass. The building with TS-3 installed meets the serviceability requirements for human perception of accelerations and almost for the torsional velocity.

To check the practical issues of using a TLD system, a parametric study is conducted on the readily variable properties of the building with TS-3 installed. The first property tested is the water height inside the TLD, which inherently affects the sloshing frequency and mass of water, simultaneously. The study demonstrates that water heights below the water height that gives a tuning ratio of unity will yield a more effective TLD design. In fact, the change in water height is likely to only benefit the system because it cannot increase, thus maintaining a near optimal efficiency. The second property studied is the amplitude modification factor, which is applied to the numerical model to calculate the physical displacement experienced by the TLD. The study essentially investigates the effect of the eccentricity of the tanks. The results show that there is a response-specific upper limit on the response reduction, which is determined by the amount of generalized water mass. Finally, the effect of varying the first and second structural frequencies on the building response is investigated. The study demonstrates that the frequency content of the wind force causes the building responses to vary without a TLD system; however, the TLD system is capable of reducing the variance and significantly reducing the responses. The responses are dependent on the tuning ratio of the TLD system, thus causing overestimations of the structural frequencies to be severely detrimental to the building; conversely, underestimations increase the TLD effectiveness.

Overall, the TS-3 design for the building is an effective and robust alternative for

controlling the coupled lateral-torsional structural motions. To improve the TLD design, an MTLD system can be used to capture a range of frequencies excited by the wind loading. This is still a relatively new concept and is an area for future research. In addition, as a means to increase the robustness of a TLD system, a post-construction dynamic survey of the building should be carried out and the TLD system should be fine-tuned to match the as-built structural frequencies. Furthermore, placing water height monitoring devices in the TLDs will significantly reduce the chance of an undesirable mistuning.

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CHAPTER 3

Reduced Equivalent Static Wind Loads for Tall Buildings with Tuned Liquid Dampers

3.1 Introduction

An accurate assessment and evaluation of the dynamic wind loads is crucial in the design of tall buildings. As building designs become taller, lighter, and more flexible and complex, there is a greater contribution from the dynamic component of the wind load. The most reliable tool for assessing the dynamic wind load is wind tunnel testing. A boundary layer wind tunnel is capable of accurately calculating an equivalent static wind load (ESWL) acting on a building. The ESWL conveniently comprises three distinct components of the wind: the stationary contribution (mean), the fluctuating contribution (background), and the contribution from dynamically exciting the building (resonant) (Solari, 1989). The mean component is directly related to the mean of the wind speed profile. The background component includes the fluctuations due to turbulence, which typically has a low range of frequencies that do not excite the building dynamically. The resonant component consists of the contribution from dynamic amplification, which occurs when the frequency of the wind matches the vibration frequencies of the building.

The most common approach to estimating the ESWL involves calculating a gust loading factor (GLF), which relates the peak response to the mean (Davenport, 1967). This method is limited to calculating the along-wind ESWL and only captures the first mode of vibration of the building. Recognizing the need to include higher vibration modes for structures that are sensitive to dynamic loading, such as high-rise buildings, Holmes (2002) proposed a formulation to accommodate higher modes to the resonant

ESWL. Moreover, the formulation utilises a previously established distribution method by Boggs and Peterka (1989) as a means to increase the accuracy of the resonant ESWL by proportionally distributing the load according to the mass and mode shape of each floor of the building.

To improve the response-specific GLF method, specifically for tall buildings, Zhou and Kareem (2001) modified the formulation from a deflection-based approach to a method that relates the response to the base bending moment (BBM). The BBM approach is more beneficial because of the following. First, it provides a more realistic representation of the cantilever action of a building. Secondly, it can accommodate nonlinear mode shapes and variations in the mass distribution along the height of a building. Finally, it is highly compatible with the high frequency force balance (HFFB) testing technique because the HFFB technique involves recording base moments for analysis. Chen and Kareem (2005) used the BBM framework to improve the resonant ESWL contribution by accounting for coupled three-dimensional modes when analysing HFFB measurements.

Accurately assessing the resonant ESWL is important for high-rise buildings because the resonant contribution can be substantially greater than the combined mean and background (quasi-steady) wind load. This is especially true for across-wind and torsional motions, which are primarily excited from dynamic excitation (Zhou et al., 1999a). Therefore, reducing the resonant wind load will significantly reduce the total wind load acting on the building. To date, dynamic vibration absorbers (DVAs) have been limited to aiding tall buildings in achieving serviceability criteria by effectively increasing the damping. By changing the dynamic properties of the building, a DVA is also capable of reducing the resonant wind loads.

A proven effective DVA is the tuned liquid damper (TLD), which utilises the sloshing motion of liquid to increase the damping of a structure. A TLD is a liquid filled (usually water) rigid tank, with a sloshing frequency that matches the natural frequency of the structure. Under dynamic excitation, the structure-TLD system oscillates, thus mobilizing the liquid. The liquid sloshes against the end walls of the tank, thereby exerting a force approximately anti-phase to the structural motion. From the inception of the nutation damper, a liquid filled disc-shaped container (Modi et al., 1990), to the modern rectangular TLD with slat screens (Tait et al., 2008b), the dynamic responses of many real structures have been reduced, including airport control towers (Fujii et al., 1990) and high-rise buildings (Hasan, 2008).

The sloshing behaviour of the TLD exhibits a hardening or stiffening effect when experiencing greater excitation, thereby leading to nonlinear properties (Reed et al., 1998). The use of damping screens inside the tank significantly reduces the nonlinearity by decreasing the wave amplitude and removing the higher sloshing harmonics. In addition, the screens increase the inherent damping of the TLD to an optimal level (Warnitchai and Pinkaew, 1998). Reducing the nonlinearity simplifies the sloshing motion and allows for a confident numerical description of the TLD. Tait et al. (2005a) developed a nonlinear numerical model to predict the sloshing motion inside a TLD and used this model to describe the TLD as an equivalent nonlinear tuned mass damper (TMD). Three similar parameters describe both the TLD and TMD: mass, frequency, and damping ratio. The equivalent TMD approach involves equating the energy dissipated by the TLD to that of a TMD under specific amplitudes of excitation; therefore, the TLD can be described by an amplitude-dependent mass, frequency, and damping ratio (Tait et al., 2004a).

This chapter investigates the reduction in the ESWL of a lateral-torsional coupled building with a TLD system installed. The building is unique because it exhibits a torsional sensitivity in the first and second structural modes of vibration. This study uses three unique multi-modal TLD systems, designed specifically for a lateral-torsional coupled building. The building ESWL is evaluated with each of the TLD arrangements using measurements from HFFB tests conducted at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario. Furthermore, the study performs a thorough parametric study to evaluate the robustness of the TLD system in reducing the ESWL. The study evaluates the influence of two practical parameters on the structure-TLD response, including the water height inside the tanks and the structural vibration frequencies. To the best of the author's knowledge, there has been no published research on the effect of a DVA on the ESWL of a structure.

3.2 ESWL Formulation

An equivalent static wind load is a set of response-specific pressures, forces, or torques such that when applied statically to a structure, the obtained response will match the peak response induced by the physical fluctuating wind. A building will have three ESWLs – one corresponding to each of the principle directions (along-wind, across-wind, and torsional). Each ESWL consists of three parts: the mean, background, and resonant components. Furthermore, each component is proportionally distributed along the height of the building according to an assumed load shape to produce the ESWL. Studies have shown that the base moment (BM) response produces the most representative ESWL (Zhou and Kareem, 2001). In other words, the BM based ESWL will give the exact BM response and good estimates of the other building responses, including the base shears

and deflections. This BM based ESWL method is well suited for the HFFB testing technique because the HFFB test involves directly recording the BMs in the sway and torsional directions (Tschanz and Davenport, 1983).

3.2.1 HFFB Technique

The HFFB testing technique is a simple cost effective tool to predict the peak wind loads acting on a building. The basis of the technique is in the estimation of the generalized wind force (GWF) from the overturning moments, $M_x(t)$ and $M_y(t)$, and base torque, $M_\theta(t)$, which are directly measured in the tests. The GWF takes the form

$$F_j^*(t) = a_{xj}\frac{1}{H}C_{xj}M_x(t) + a_{yj}\frac{1}{H}C_{yj}M_y(t) + 0.7a_{\theta j}C_{\theta j}M_\theta(t) \tag{3.1}$$

where $C_{\gamma j}$ is the mode shape correction factor and $a_{\gamma j}$ is the modal mixing factor for direction γ (where $\gamma = x, y, \theta$) and mode j. The base moments from the HFFB test are measured at a set of axes located at the center of coordinates (CC) of the model and are adjusted to full-scale. The HFFB technique requires a unified coordinate system for measurements and calculations named the CC. Unlike the center of masses (CM), the CC location is independent of the storey. To align the building coordinate system for proper analysis, the mode shapes require transformation from the CM to the CC according to

$$\begin{Bmatrix}\varphi_{xij}\\ \varphi_{yij}\\ \varphi_{\theta ij}\end{Bmatrix}_{\text{coordinate}} = \begin{bmatrix}1 & 0 & e_{\text{yi}}\\ 0 & 1 & -e_{\text{xi}}\\ 0 & 0 & 1\end{bmatrix}\begin{Bmatrix}\varphi_{xij}\\ \varphi_{yij}\\ \varphi_{\theta ij}\end{Bmatrix}_{\text{mass}} \tag{3.2}$$

where e_{yi} is the eccentricity from the CM to the CC for floor i. The new CC mass matrix must satisfy

$$[m]_{\text{mass}}\{\varphi^2\}_{\text{mass}} = [m]_{\text{coordinate}}\{\varphi^2\}_{\text{coordinate}} \tag{3.3}$$

The transformation has inherent benefits, including linearizing the mode shapes (Tse et

al., 2009) and assembling the lumped storey masses on a single vertical axis, which simplifies the analysis. Furthermore, the transformation does not alter the generalized properties of the building (Yip, 1995).

The basic assumption in the GWF formulation is that the sway mode shapes are linear and the torsional mode shape is constant. The mode shape correction factors account for any nonlinearity in the mode shapes and the 0.7 empirical factor relates the torsional mode shape to a constant value (Holmes et al., 2003). The power law exponents of the mode shape and mean wind speed profile affect the correction factor (Lam and Li, 2009). However, the mean wind speed profile causes minimal change in the correction factor, thus is generally ignored (Zhou et al., 1999b). Finally, the modal mixing factors account for the relative contribution from lateral-torsional coupling and are equal to the mode shape value at the top floor based on the CC (Yip and Flay, 1995).

3.2.2 Base Moments

There are two methods to evaluating the ESWL of the building; the first method uses the frequency domain to evaluate the ESWL using a statistical peak factor; the second method uses the time domain to evaluate the ESWL. Both methods accurately predict the BMs for the mean, background, and resonant responses. The solution of the TLD interaction with the building requires analysis in the time domain; therefore, the following strictly describes the time domain BM formulation.

The time domain approach involves formulating each BM component of the wind as a function of time and then calculating the peak BM through extreme-value statistical analysis. The combined action of the mean and background BMs is called the quasi-steady BM, which is the component of the turbulent wind with frequencies that are too

low to excite the building dynamically. The mean BM, $\bar{M}_\gamma$, is simply the mean of the HFFB time history recorded BM adjusted to full-scale for direction γ. The background BM, $M_{B\gamma}(t)$, is the difference of the HFFB time history record and the mean, $\bar{M}_\gamma$. The resonant BM consists of the contribution from dynamic amplification, which occurs when the frequencies in the wind match the frequencies of the building. Accounting for the first three fundamental modes is typically sufficient. The resonant BM, $M_{R\gamma j}(t)$, for mode j acting at the CC is given by

$$\begin{Bmatrix} M_{Rxj}(t) \\ M_{Ryj}(t) \\ M_{R\theta j}(t) \end{Bmatrix} = \ddot{\xi}_{sj}(t) \sum_i \begin{Bmatrix} z_i \\ z_i \\ 1 \end{Bmatrix}^T \begin{bmatrix} m_i & 0 & -e_{yi}m_i \\ 0 & m_i & e_{xi}m_i \\ -e_{yi}m_i & e_{xi}m_i & I_i + \left({e_{xi}}^2 + {e_{yi}}^2\right)m_i \end{bmatrix} \begin{Bmatrix} \varphi_{xij} \\ \varphi_{yij} \\ \varphi_{\theta ij} \end{Bmatrix} \tag{3.4}$$

where $\ddot{\xi}_{sj}(t)$ is the generalized acceleration of the structure, which is obtained by solving the classical single degree-of-freedom (SDOF) system formulated as

$$m_j^* \ddot{\xi}_{sj}(t) + c_j^* \dot{\xi}_{sj}(t) + k_j^* \xi_{sj}(t) = F_j^*(t) \tag{3.5}$$

where m_j^*, c_j^*, and k_j^* are the building generalized mass, damping coefficient, and stiffness of the building excited by the GWF, $F_j^*(t)$. Determination of the generalized acceleration of the structure with a TLD system installed is discussed in a subsequent section.

The use of a single peak from the temporal responses (background and resonant) will likely cause an overestimation of the BMs because of outlying peaks. In other words, one peak (an outlier) in the temporal response may be significantly greater than the other peaks, thereby causing an overestimation of the BM. To avoid overestimating the BM, a statistical analysis of the peak BMs, or extreme values, is necessary. There is a difference between the statistical distributions that govern extreme-value data and ordinary data.

Generally, ordinary data follows a normal or Gaussian distribution. However, extreme-value data, with a random sample, n, from a population, typically follows a Type I extreme-value distribution described by the cumulative distribution function

$$P(X \leq x) = e^{-e^{-\frac{(x-u)}{b}}} \tag{3.6}$$

where u is the location parameter and b is the dispersion parameter. Lieblein (1974) developed a procedure to determine the location and dispersion parameters of the distribution called the Lieblein BLUE Technique for three cases: a sample size of $n \leq 16$, $16 < n \leq 50$, and $n > 50$.

Dividing the time series of the BM components into n intervals and calculating the extreme value in each interval provides an array of points that follow a Type I extreme-value distribution. The 1-hour peak BM is subsequently calculated by

$$\widetilde{M} = u + (ln(N) + 0.577)b \tag{3.7}$$

where u and b are the statistical parameters for the base moment $\widetilde{M}$, which corresponds to either the background, $\widetilde{M}_{B\gamma}$, or resonant, $\widetilde{M}_{R\gamma j}$, component. Also, N is the number of intervals from n such that the sample time is equal to 1 hour from the entire time history (population). The confidence interval (CI) is dependent on the sample size N. For example, $N = 10$ (a sample every 6 minutes) provides a CI of approximately 95%.

3.2.3 Load Shapes

Effective load shapes (ELSs) are used to distribute the bending moments along the height of the building. Each component of the wind has a unique distribution; however, for simplicity, the mean, $\bar{\chi}_{\gamma i}$, and background, $\tilde{\chi}_{B\gamma i}$, are assumed to follow the same shape, called the quasi-steady ELS. Normally, the quasi-steady ELS is obtained by

pressure integration over the height of the building (Boggs and Peterka, 1989); however, this data is not available from HFFB tests. Therefore, the formulation assumes a trapezoidal pressure shape with the top and bottom magnitudes determined by resolving the peak hourly quasi-steady base shear, $\hat{S}_{Q\gamma}$, and moment, $\hat{M}_{Q\gamma}$, according to

$$p_{\gamma i} = \left(4 - 6\frac{z_i}{H}\right)\frac{\hat{S}_{Q\gamma}}{H} - \left(6 - 12\frac{z_i}{H}\right)\frac{\hat{M}_{Q\gamma}}{H^2} \tag{3.8}$$

for floor i in the sway directions ($\gamma = x, y$) and where H is the building height. The ELSs are then proportional to

$$\begin{Bmatrix} \tilde{\chi}_{xi} \\ \tilde{\chi}_{yi} \\ \tilde{\chi}_{\theta i} \end{Bmatrix} = \begin{Bmatrix} \tilde{\chi}_{Bxi} \\ \tilde{\chi}_{Byi} \\ \tilde{\chi}_{B\theta i} \end{Bmatrix} \propto \begin{Bmatrix} p_{xi} B_{xi} h_i \\ p_{yi} B_{xi} h_i \\ \dfrac{(p_{xi} + p_{yi})}{2} \sqrt{B_{xi} B_{yi}} \end{Bmatrix} \tag{3.9}$$

where B_{yi} is the projected building width and h_i is the storey height. The quasi-steady ELSs are normalized to unit base moments and are dependent on the wind direction since the peak quasi-steady base shear and moment vary with wind direction.

The resonant ELSs, $\tilde{\chi}_{R\gamma j}$, are simply proportional to the distribution of the mass and mode shapes of the building at the CC given by

$$\begin{Bmatrix} \tilde{\chi}_{R\gamma ij} \\ \tilde{\chi}_{R\gamma ij} \\ \tilde{\chi}_{R\gamma ij} \end{Bmatrix} \propto \begin{bmatrix} m_i & 0 & -e_{yi} m_i \\ 0 & m_i & e_{xi} m_i \\ -e_{yi} m_i & e_{xi} m_i & I_i + \left({e_{xi}}^2 + {e_{yi}}^2\right) m_i \end{bmatrix} \begin{Bmatrix} \varphi_{xij} \\ \varphi_{yij} \\ \varphi_{\theta ij} \end{Bmatrix} \tag{3.10}$$

which are normalized to unit base moments. Note that the resonant ELSs are independent of the wind direction, thus are only solved once.

Using the ELSs combined with the BMs, each wind component has a unique distribution of forces or torques along the height of the building. The background and resonant components are combined using the square root of the sum of squares (SRSS) method (Holmes, 2002). Then the mean component is added to give the total ESWL.

The ESWL for floor i is given by

$$\hat{F}_{Yi} = \sum_{\alpha} \beta(\alpha) \left(\bar{M}_Y(\alpha)\bar{\chi}_{Yi}(\alpha) + \sqrt{\left(\tilde{M}_{BY}(\alpha)\tilde{\chi}_{BYi}(\alpha)\right)^2 + \sum_{j} \left(\tilde{M}_{RYj}(\alpha)\tilde{\chi}_{RYij}\right)^2} \right) \quad \textbf{(3.11)}$$

where $\beta(\alpha)$ is the relative importance factor for a given wind angle of incidence. The HFFB technique involves testing the model at various wind angles, α, and combining them through importance factors, $\beta(\alpha)$. These factors account for the relative contribution from each wind direction by means of the climate and aerodynamic directionality, which are calculated using site-specific meteorological data. The summation of the importance factors is unity.

3.3 ESWL for the Test Building

The current study developed a numerical model to formulate the ESWL for a building from HFFB wind test data. The model utilises the previously described ESWL formulation. The SDOF equation of motion (equation 3.5) uses the fourth order Runge-Kutta-Gill approach to obtain a solution through iterations in the time domain. Then the Lieblein BLUE technique is used to obtain a peak hourly acceleration from the time history solution. The BLWTL provided the numerical code for the Lieblein BLUE technique. In addition, the numerical model establishes the ELSs for the ESWL calculation. The numerical model solves the ESWL for many wind directions and combines them using the site-specific wind climate model. The ESWLs for the following test building are obtained using this numerical model.

3.3.1 Building Description

The building tested in this study is 50 stories tall (160.3m) and uses reinforced concrete shear walls and moment resisting frames (MRFs) to resist the lateral loads. Figure 3.1 shows the irregular L-shaped floor plan along with the chosen CC and the sign conventions. The MRFs are located along the east and south exterior faces of the building with the shear walls predominantly in the core. The mass distribution (Figure 3.2a), mass moment of inertia distribution (Figure 3.2b), and eccentricities (Figure 3.2c) along the height of the building are important for determining the resonant ESWL. The relatively large mass at the top floor is due to higher dead loads from mechanical equipment and the additional structural support system for the TLDs.

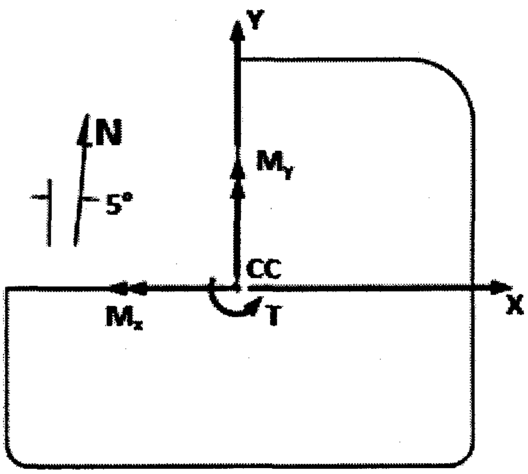

Figure 3.1 A schematic showing the center of coordinates and the sign conventions.

From modal analysis, Figure 3.3 shows highly coupled action of the first three vibration modes, which have corresponding vibration periods of 6.30, 5.30, and 3.78 seconds. The torsional mode shapes are multiplied by the overall radius of gyration (13.2m) of the building to maintain dimensional consistency with the sway mode shapes. In other words, this technique allows for evaluation of the relative contribution from each

direction. In addition, the mode shapes are evaluated at the CC according to the relationship described by equation 3.2.

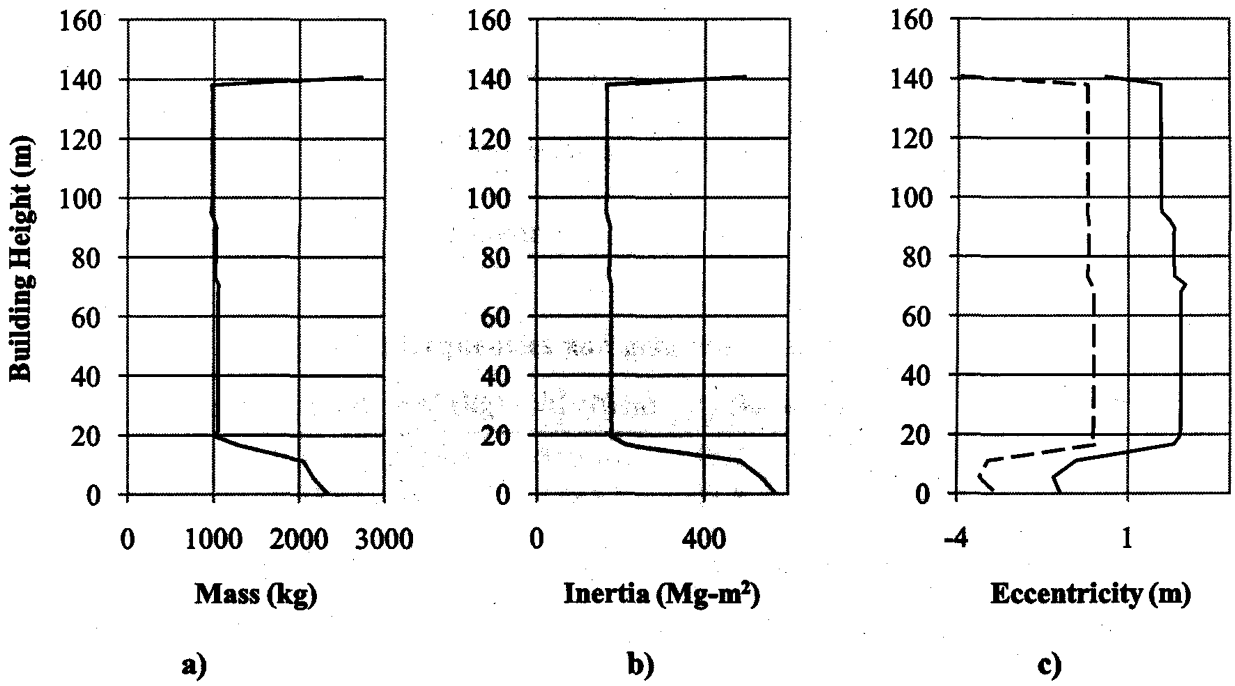

Figure 3.2 The storey a) mass and b) mass moment of inertia and c) eccentricity in the X (solid line) and Y (dashed line) directions of the building.

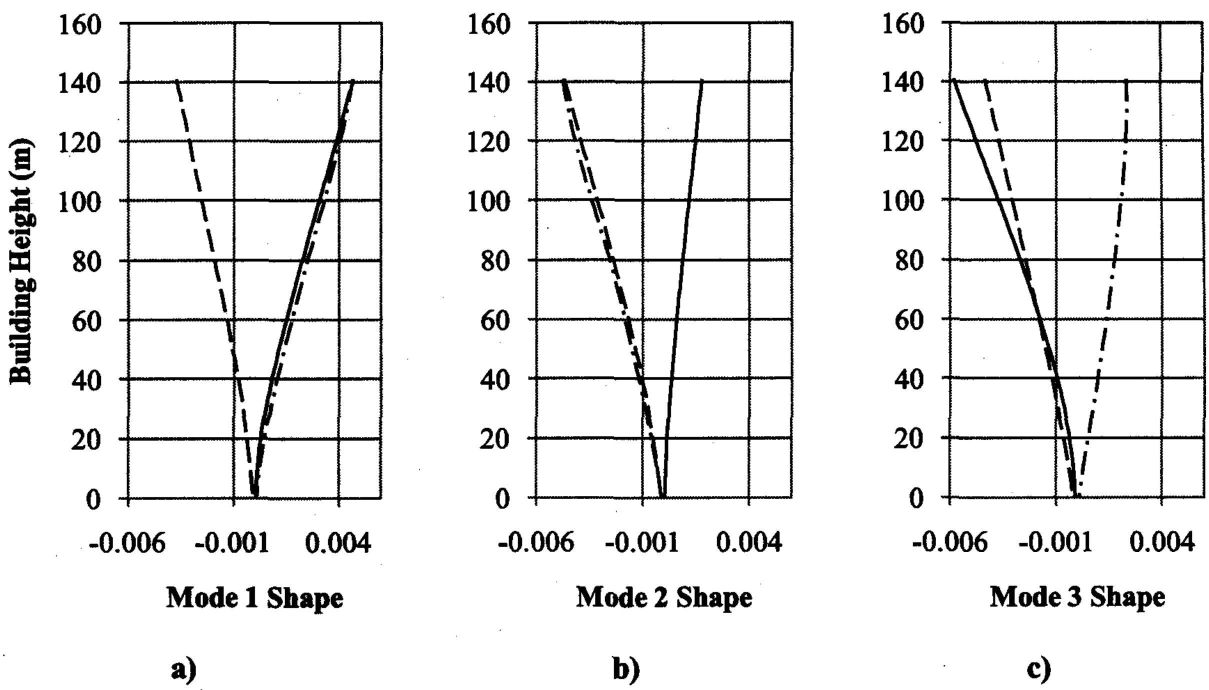

Figure 3.3 The mode shapes for sway in the X direction (solid line), sway in the Y direction (dashed line), and torsion (dash-dot line) for a) mode 1, b) mode 2, and c) mode 3.

Table 3.1 shows the generalized properties required for analysis, which are calculated from the mass distributions and dynamic characteristics of the building. Using the exact damping ratio is crucial in the design stages of a structure; therefore, to reduce the uncertainty, 1% and 2% damping ratios are implemented in the analysis. However, for this building, the 2% damping ratio is a more likely estimate because of the increased motion due to strong design wind loads.

Table 3.1 The modal frequencies and generalized properties of the building.

Mode j	**f_j (Hz)**	**m_j^* (kg)**	**k_j^* (N/m)**	**c_j^* (kg/s) (1%)**	**c_j^* (kg/s) (2%)**
1	0.1588	988.8	984.0	19.7	39.5
2	0.1887	983.6	1383.2	23.3	46.7
3	0.2645	978.1	2700.9	32.5	65.0

3.3.2 Wind Loading

The ESWL is predicted using wind tunnel data from tests conducted at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario. The test data was recorded using a high frequency force balance (HFFB) with a 1:400 scale rigid lightweight foam model mounted on it. The surrounding buildings were modelled within a 550-meter radius and cubic roughness elements were raised to replicate the upwind terrain. The model was tested every 10 degrees of azimuth and at a few other critical wind angles and exposures for a sum of 43 tests. Each test lasted 250 seconds (approximately 4 hours in full-scale) and data were sampled at a rate of 116 hertz. Applying the site-specific meteorological wind climate model to the test data gives base moments that are used for calculating the GWF. The wind climate model for the building in this study predicts a mean hourly wind speed of 46.8m/s at 500 meters for a 50-year return period – the return period for strength design.

Using the time domain ESWL formulation, effective wind loads in the X, Y, and θ directions are solved for each test and combined using the response-specific importance factors. Table 3.2 shows the mean (M) and background (B) BMs and the resonant BMs for mode 1 (R1), mode 2 (R2), and mode 3 (R3) for 1% and 2% damping ratios. In addition, Table 3.2 includes the percent contribution from each component of the wind. The main purpose of this statistic is to quantify the resonant contribution, which is the portion that a DVA is capable of controlling. The contribution is greater than 50 percent, which is significant due to the torsional sensitivity of the building. The damping ratio should not affect the mean and background BMs; however, the table shows a trivial difference because slightly different importance factors, β, are used for the two damping cases. A second observation is the relative contributions from each of the modes. The first mode contribution clearly demonstrates the highly coupled motion of the building. Nonetheless, each mode contributes to all three directions because of the lateral-torsional coupling action. However, there is a less significant contribution from mode 2 and mode 3 in the X direction and θ direction, respectively. These resonant contributions are directly related to the relative mode shape magnitudes shown in Figure 3.3.

Figure 3.4 illustrates each component of the ESWL from Table 3.2 along the height of the 2% damped building according to the ELSs. Noticeably, the mean and background components are distributed according to the same load shape. The top floor ESWL for the background component is relatively large due to the greater surface area in which the pressure is acting. The greater surface area is due to the 21-meter storey height for the top floor. Similarly, the resonant components are relatively large at the top floor because the associated dead load is substantially greater. Similarly, the bottom floors attract higher load because of a greater projected width and consequently additional mass.

Table 3.2 The base moments for each wind load component at 1% and 2% damping ratios.

BM	Component	1% Damping		2% Damping	
		Peak	**% of Total BM**	**Peak**	**% of Total BM**
X (kN-m)	**M**	373880	39.1	422902	44.2
	B	134615	14.1	187786	19.6
	R1	276339	28.9	211144	22.1
	R2	16838	1.8	12820	1.3
	R3	144728	15.1	111379	11.6
Y (kN-m)	**M**	145559	19.6	164569	22.1
	B	209375	28.1	286377	38.5
	R1	105288	14.2	78686	10.6
	R2	196906	26.5	145912	19.6
	R3	75980	10.2	57722	7.8
T (kN-m)	**M**	20122	20.1	24481	24.5
	B	4690	4.7	7571	7.6
	R1	33231	33.2	29673	29.6
	R2	37561	37.5	34049	34.0
	R3	3378	3.4	2996	3.0

Generally, for regular buildings, the across-wind and torsional components have a mean component of approximately zero and a relatively low background and resonant components (Zhou et al., 1999a). The site-specific dominant wind direction approximately aligns with the *X* direction of the building, thus the *Y* direction corresponds to the across-wind direction. As expected, the mean is noticeably less significant in the approximate across-wind (Figure 3.4b) and torsional (Figure 3.4c) directions. Furthermore, the background component contributes minimally to the torsional direction, thereby facilitating the dominance of the resonant portion. Figure 3.4c illustrates the significant torque contribution from mode 1 and mode 2, which is the primary reason to design a multi-modal TLD system. The highly lateral-torsional coupled action of the building leads to these significant resonant contributions in both the across-wind and torsional BMs.

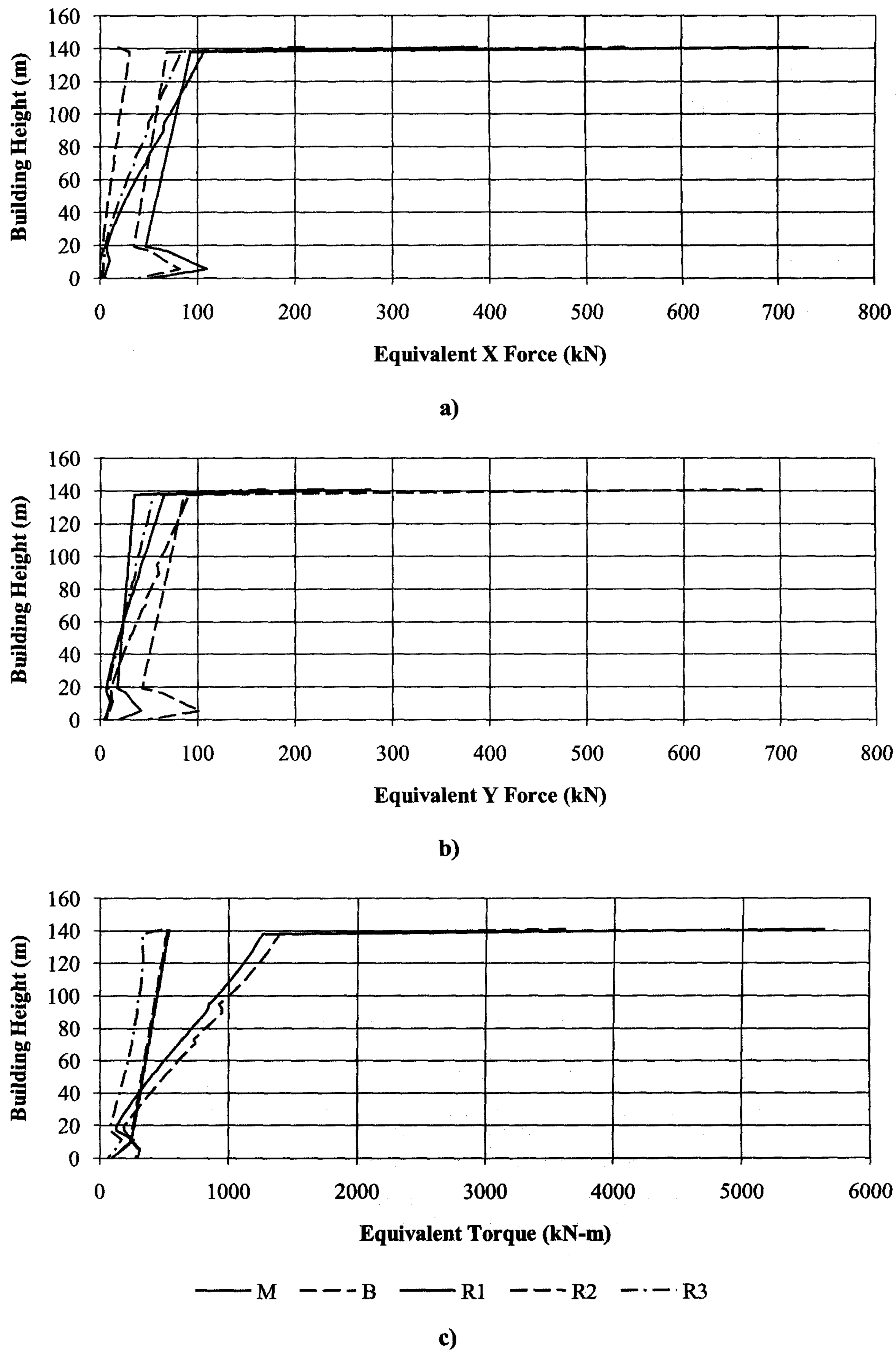

Figure 3.4 The relative distributions from each wind load component for the a) X direction, b) Y direction, and c) torsion.

Figure 3.5 combines the components in Figure 3.4 according to equation 3.11, to illustrate the total ESWLs for the 2% damped building. The building design is governed by this wind loading. Table 3.3 shows the peak hourly resultant base shears and bending moments induced by the ESWL with the corresponding root-mean-square (RMS) values. The RMS values are a measure of the variance in the time history response. The RMS values cannot be calculated for the base shears because the ESWL is a constant magnitude that does not vary temporally. The base reactions in the *X* direction are greater than the *Y* direction because of the lower quasi-steady contribution in the across-wind direction. The high resonant contribution to the torsional motion causes the damping ratio to have a greater effect on the base torque than the overturning moments. To reiterate, applying the ESWL to the building will yield the predicted hourly peak BMs shown in Table 3.3 and the approximate hourly peaks for all other responses, including the base shears and top floor deflections.

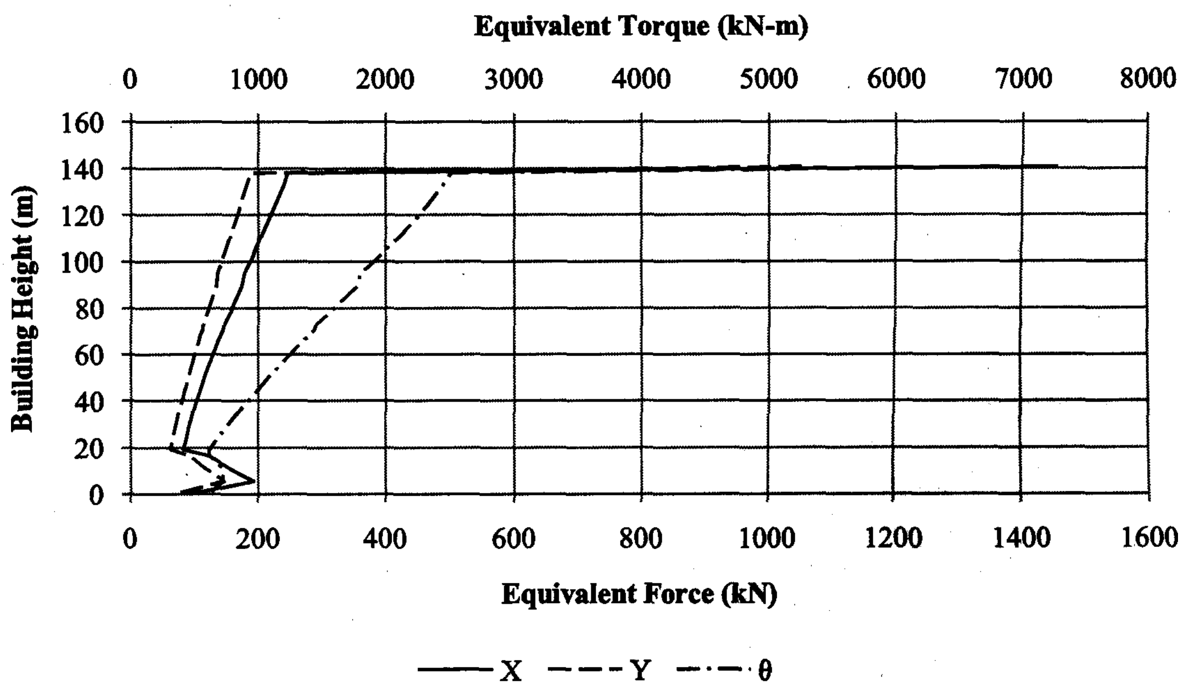

Figure 3.5 The ESWLs for the three principle directions.

Table 3.3 The base reactions resulting induced by the ESWLs.

Base Reaction	1% Damping		2% Damping	
	Peak	RMS	Peak	RMS
X Shear (kN)	10190	-	9148	-
Y Shear (kN)	7986	-	6967	-
X Bending Moment (kN-m)	956539	297435	847068	234679
Y Bending Moment (kN-m)	743922	298598	641947	242897
Torsional Moment (kN-m)	100082	40358	80422	29099

3.4 Implementation of Tuned Liquid Dampers

A tuned liquid damper is a device that is capable of effectively controlling the dynamic responses of a building. The primary purpose of a TLD is to control serviceability limit responses, such as excessive accelerations and inter-storey drifts. However, by reducing the accelerations experienced by a building, the TLD is also reducing the inertial loading on the structure. This results in lower base shears, bending moments, and torques. Therefore, if a large proportion of the ESWL is from the resonant component, a TLD will effectively reduce the wind loading. The building in this study is ideal for verifying this concept because, as shown in Table 3.2, the resonant portion of the ESWL is significant.

3.4.1 Behaviour

The water sloshing frequency, participating water mass, and damping ratio characterize the behaviour of a TLD. The main factors influencing these parameters are the tank dimensions and screen configuration, as shown in Figure 3.6. Despite the nonlinear behaviour of a TLD, the use of linear wave theory to estimate the sloshing frequency, f_w, and the potential flow theory to estimate the water mass contributing to the

fundamental sloshing mode, m_1, are acceptable for the initial design (Tait et al., 2005a). These initial design equations take the form

$$f_w = \frac{1}{2\pi}\sqrt{\frac{\pi g}{L_w}\tanh\left(\frac{\pi h_w}{L_w}\right)} \tag{3.12}$$

$$\alpha_p = \frac{m_1}{m_w} = \frac{8\tanh\left(\frac{\pi h_w}{L_w}\right)}{\pi^3\left(\frac{h_w}{L_w}\right)} \tag{3.13}$$

where g is the acceleration of gravity, h_w is the water height, L_w is the tank length in the direction of excitation, and m_w is the mass of the water inside the tank.

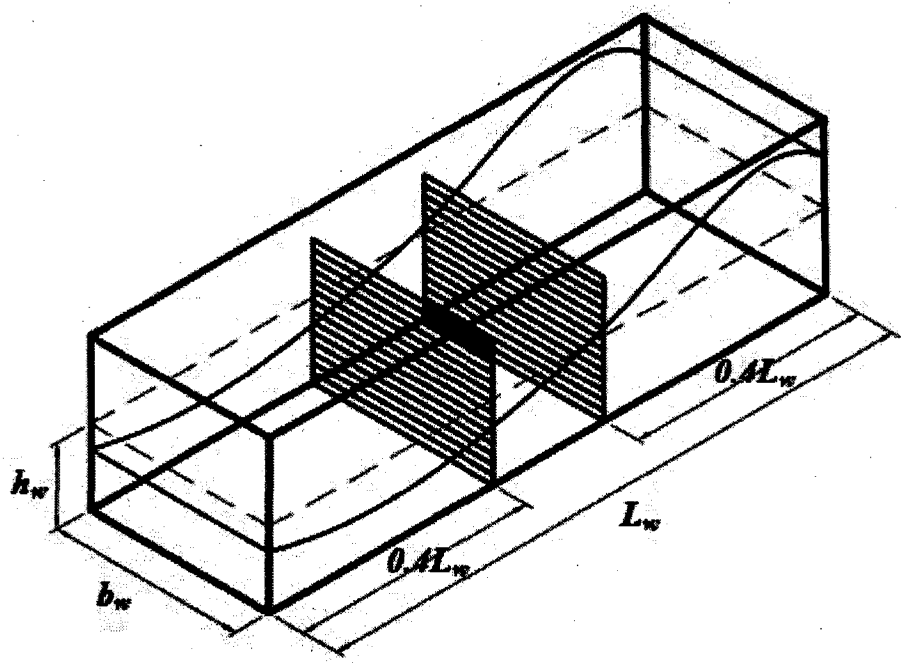

Figure 3.6 The dimensions of a 1D TLD and the placement of the screens.

The function of a TLD is to suppress the dynamic motions of a desired mode. The ratio of the sloshing frequency to the structural frequency of that desired mode is called the tuning ratio, Ω. This ratio, in conjunction with the mass ratio, μ, strongly influences the TLD performance. The mass ratio is the ratio of the total generalized participating water mass to the structural generalized mass. These ratios take the form

$$\Omega = \frac{f_w}{f_s} \tag{3.14}$$

$$\mu = \frac{\alpha_p M_p}{m^*} \tag{3.15}$$

where α_p is the proportion of water mass that contributes to the fundamental sloshing mode of the TLD defined by equation 3.13 and m^* is the generalized mass of the building for the desired mode. M_p is the total generalized water mass defined by equations 3.16 and 3.17. To promote an optimal TLD design, set the tuning ratio to near unity and the mass ratio to within a range of one to four percent (Tait, 2008a).

For the purpose of analysis, the three TLD properties must be expressed as functions of the excitation amplitude. This is possible through the equivalent tuned mass damper (TMD) approach. The technique involves equating the energy dissipated by the TLD to an equivalent amplitude-dependent TMD. The result is an amplitude-dependent expression for the frequency, f_{TLD}, mass, m_{TLD}, and damping ratio, ζ_{TLD}. Depending on the tank dimensions and screen configuration, these expressions fit either a linear or a power function.

3.4.2 Unique Designs

The building in this study requires a unique TLD system designed specifically for the lateral-torsional coupled motion. Figure 3.7 shows the three multi-modal TLD system designs. Herein, the systems are labelled TS-1, TS-2, and TS-3. TS-1 uses 1D tanks, TS-2 utilises 2D tanks, and TS-3 employs two layers of 1D tanks. One-dimensional tanks reduce motion in a single direction along the length of the tank while two-dimensional tanks are capable of reducing motion in both orthogonal directions. The 2D tanks are designed as two independent 1D tanks. This is possible because the wave motions and

base shear forces in the tanks are uncoupled in the two orthogonal directions (Tait et al., 2005b).

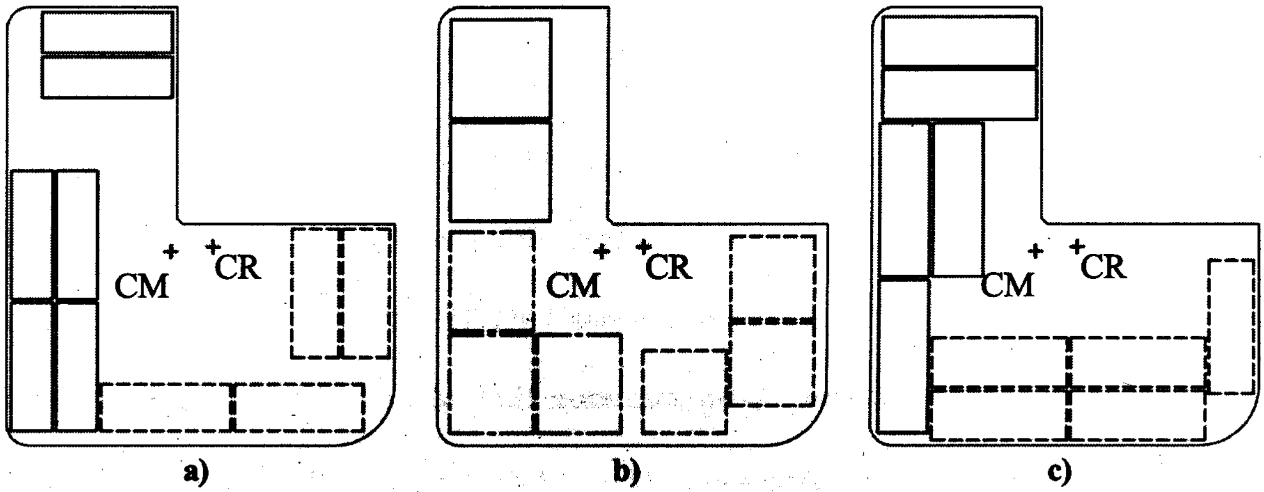

Figure 3.7 The layout of the TLDs tuned to mode 1 (solid line), mode 2 (dashed line), and both mode 1 and mode 2 (dash-dot line) for a) TS-1, b) TS-2, and c) TS-3.

All three TLD systems are designed to be installed on the roof (z = 140.7m) of the building. Li et al. (2004) demonstrated that designing TLDs for the first several modes will significantly improve the effectiveness of the vibration control; therefore, each TLD system has two sets of tanks – one set tuned to mode 1 and a second set tuned to mode 2. Figure 3.7 shows the placement of each set of tanks around the perimeter of the floor plan. This maximizes the displacement experienced by the TLDs, thereby increasing the effectiveness of the TLD system. In addition, the tank placement increases the generalized water mass, m_p, by introducing an eccentricity, L_x or L_y, between the tanks and the center of mass of the roof in the X and Y directions, respectively. The generalized water mass for a tank aligned in the X or Y directions, respectively, are

$$m_p = m_w(\varphi_x - L_y\varphi_\theta)^2 \tag{3.16}$$

$$m_p = m_w(\varphi_y + L_x\varphi_\theta)^2 \tag{3.17}$$

where φ_Y is the mode shape corresponding to the mode in which the tank is tuned.

The total generalized water mass, M_p, is the summation from each individual tank. Table 3.4 summarizes the design of each TLD system in terms of the tuning ratio, mass ratio, and optimal damping. The main difference between the designs is the generalized water mass, which consequently affects the TLD mass ratio, μ. The TLD system layouts are chosen to capture a wide range of TLD mass ratios such that TS-2 has a TLD mass ratio approximately twice that of TS-1 and two-thirds that of TS-3. This allows for the investigation on the effect of the TLD mass ratio on the building responses. Note that the physical mass ratio, M_w/m_s, where M_w is the total water mass and m_s is the structure mass, is significantly less than the TLD mass ratio because the TLD mass ratio accounts for the mode shape and eccentricity of the tanks. In addition, despite the similar water mass between TS-2 and TS-3, TS-3 uses the floor space more effectively, thereby resulting in a higher TLD mass ratio.

Table 3.4 The tank dimensions, sloshing frequency, mass characteristics, and optimal effective damping for the three TLD systems.

	Mode	L_w (m)	h_w (m)	f_w (Hz)	α_p	M_p (kg)	M_w/m_s (%)	μ (%)	$\zeta_{eff-opt}$
TS-1	1	10.4	1.157	0.1588	0.779	16.3	0.37	1.28	0.0283
	2	10.4	1.708	0.1887	0.746	19.3	0.42	1.46	0.0303
TS-2	1	8.0	0.673	0.1588	0.792	37.2	1.15	2.98	0.0433
	2	6.7	0.673	0.1887	0.785	45.8	1.14	3.65	0.0480
TS-3	1	12.4	1.674	0.1588	0.765	54.6	1.30	4.22	0.0516
	2	10.8	1.855	0.1887	0.740	65.7	1.25	4.94	0.0559

The role of the damping screens inside the tanks is to increase the inherent damping of the TLD (Fediw et al., 1995). To achieve a near optimal effective damping ratio (see Table 3.4), two slat screens with a solidity ratio of 0.42 are placed at $0.4L_w$ and $0.6L_w$.

The effective optimal damping is based on an equivalent linear TMD and is given by

$$\zeta_{eff-opt} = \frac{1}{4}\sqrt{\frac{\mu + \mu^2}{1 + \frac{3\mu}{4}}} \quad (3.18)$$

where μ is the TLD mass ratio (Luft, 1979). The use of slat screens significantly reduces the nonlinearity of the TLD by removing the higher sloshing harmonics and reducing the maximum wave height (Tait et al., 2005a). Furthermore, to avoid the complications from wave breaking, the wave height is kept below $2h_w$, which defines, approximately, when wave breaking will occur (Sun and Fujino, 1994).

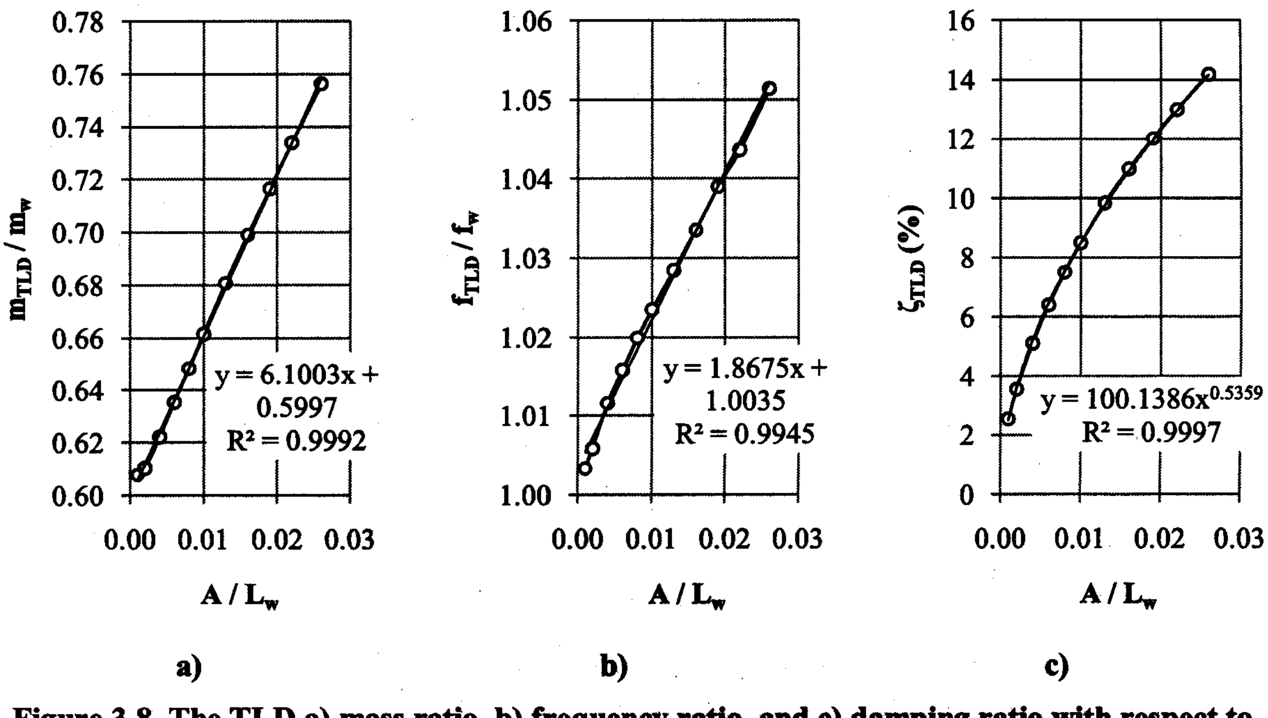

Figure 3.8 The TLD a) mass ratio, b) frequency ratio, and c) damping ratio with respect to the normalized amplitude of excitation for mode 1 tanks of TS-3.

The numerical model developed by Tait et al. (2005a) is capable of checking the wave height restriction. The model also solves the amplitude-dependent properties of the TLD by performing discrete frequency sweep tests and using the equivalent TMD approach. Figure 3.8 shows the equivalent amplitude-dependent TMD mass, frequency, and

damping ratio for the mode 1 tanks of TS-3 with the corresponding fitted linear or power curve. The R^2 value is a measure of how well the curve fits with the data points (unity is a perfect fit). Mode 1 and mode 2 tanks of each TLD system have a unique set of these properties, which are required for the evaluation of the ESWL.

3.5 ESWL Reduction

3.5.1 Structure-TLD Analysis

Tait et al. (2004b) formulated a method to analyse the complex structure-TLD system with the TLD tuned to a particular mode. Figure 3.9b illustrates the building characterized by a generalized mass, m_j^*, stiffness, k_j^*, and damping, c_j^*, for mode j. The TLD interacts with the generalized building through an auxiliary degree-of-freedom (DOF) with amplitude-dependent equivalent TMD properties as shown in Figure 3.9c. The equation of motion for the resulting 2DOF system takes the form

$$\begin{bmatrix} m_j^* + m_o & 0 \\ 0 & m_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \ddot{\xi}_s(t) \\ \ddot{\xi}_{TLD}(t) \end{Bmatrix} + \begin{bmatrix} c_j^* + c_{TLD}(A) & -c_{TLD}(A) \\ -c_{TLD}(A) & c_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \dot{\xi}_s(t) \\ \dot{\xi}_{TLD}(t) \end{Bmatrix} + \begin{bmatrix} k_j^* + k_{TLD}(A) & -k_{TLD}(A) \\ -k_{TLD}(A) & k_{TLD}(A) \end{bmatrix} \begin{Bmatrix} \xi_s(t) \\ \xi_{TLD}(t) \end{Bmatrix} = \begin{Bmatrix} F_j^*(t) \\ 0 \end{Bmatrix} \qquad (3.19)$$

where m_{TLD}, k_{TLD}, *and* c_{TLD}, are the equivalent nonlinear TMD mass, stiffness, and damping coefficient with respect to the amplitude, A, respectively. The excitation force is the GWF, $F_j^*(t)$, from equation 3.1. Equation 3.19 is solved using the fourth-order Runge-Kutta-Gill method, which accommodates the nonlinear TLD property equations through an iterative time-dependent process. The process involves initializing the problem with trivial values for m_{TLD}, k_{TLD}, *and* c_{TLD}. Using these values, the responses for the first cycle, n, are calculated and the absolute maximum generalized displacement

is obtained. This displacement is transformed to the physical displacement experienced by the TLD through an amplitude modification factor (AMF), Φ_j. The AMF accounts for the eccentricity between the TLD and the CM of the building. The factor is a normalized weighted-average of the generalized water mass of the tank multiplied by the mode shape value at the tank location. Using the physical displacement from cycle n as the amplitude, A, the updated $m_{TLD}, k_{TLD}, and\ c_{TLD}$ for cycle $n + 1$ are calculated using the TLD property equations (Figure 3.8). This iterative process continues until the time-dependent GWF reaches the end. This analysis methodology is used to calculate the responses for mode 1 and mode 2. Mode 3 is solved as an SDOF system because there are no TLDs tuned to the third mode. The three modes are combined through addition, thereby resulting in a time-dependent structural acceleration for the ESWL formulation.

The current study introduces the AMF into the previously described numerical model, which was developed by Tait et al. (2004b). The AMF is required because there are many TLDs installed at different eccentricities to the center of mass of the building. Since each TLD has a different eccentricity, each TLD experiences a slightly different displacement (or amplitude) due to the twist of the building. Therefore, the AMF is introduced as a means to calculate the average amplitude experienced by the TLD system. This essentially simplifies the TLD system by creating a single lumped TLD, thereby allowing a representation of the TLD by a single auxiliary DOF (Figure 3.9). This is possible because the sloshing frequency, damping ratio, and the TLD property equations for each TLD are the same. If any of these properties are different, then separate DOFs are required.

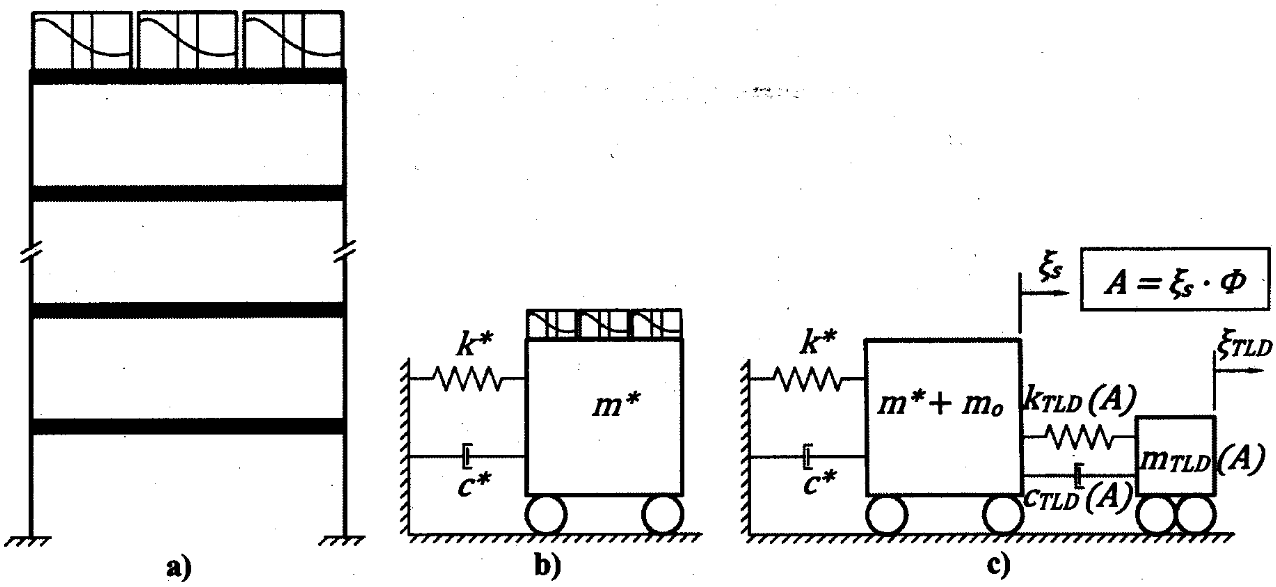

Figure 3.9 The evolution of a) the structure-TLD system into b) a generalized structural system with TLDs then into c) a system with equivalent TMD representation.

The peak hourly base moments and shears for the building with each of the three TLD systems are shown in Table 3.5 and Table 3.6 for 1% and 2% damping ratios, respectively. The percent reduction, Ψ, is shown in brackets below the peak shears and moments and is calculated by

$$\Psi = \frac{R_{No\ TLD} - R_{With\ TLD}}{R_{No\ TLD}} \cdot 100 \tag{3.20}$$

where $R_{With\ TLD}$ and $R_{No\ TLD}$ are the responses with and without a TLD system installed, respectively. The reduction evidently increases as the TLD mass ratio increases, thereby indicating the importance of the amount of generalized water mass. As expected, the percent reductions for the base responses are less significant than for the serviceability responses, such as deflections and accelerations. This is because the base responses include a significant contribution from the quasi-steady component (see Table 3.2). The TLD systems are only capable of controlling the resonant components; therefore, the reductions in the base responses are limited, unlike for the serviceability responses. This limits the effectiveness of the TLD system at reducing the strength design responses;

however, both TS-2 and TS-3 remain relatively effective, especially for the building with a 1% damping ratio. There is a noticeably greater reduction in the base torque because the resonant component is dominant.

Table 3.5 The base reactions for the structure-TLD systems with a 1% damping ratio.

Base Reaction	TS-1		TS-2		TS-3	
	Peak	RMS	Peak	RMS	Peak	RMS
X Shear (kN)	9439	-	9230	-	9153	-
	(7.4)	-	(9.4)	-	(10.2)	-
Y Shear (kN)	7067	-	6889	-	6843	-
	(11.5)	-	(13.7)	-	(14.3)	-
X Moment (kN-m)	877420	249170	855237	237777	847023	235210
	(8.3)	(16.2)	(10.6)	(20.1)	(11.4)	(20.9)
Y Moment (kN-m)	650482	244831	632111	235513	627321	236850
	(12.6)	(18.0)	(15.0)	(21.1)	(15.7)	(20.7)
Torsion (kN-m)	77987	27072	72092	24085	70266	24004
	(22.1)	(32.9)	(28.0)	(40.3)	(29.8)	(40.5)

Table 3.6 The base reactions for the structure-TLD systems with a 2% damping ratio.

Base Reaction	TS-1		TS-2		TS-3	
	Peak	RMS	Peak	RMS	Peak	RMS
X Shear (kN)	8833	-	8689	-	8625	-
	(3.4)	-	(5.0)	-	(5.7)	-
Y Shear (kN)	6571	-	6459	-	6412	-
	(5.7)	-	(7.3)	-	(8.0)	-
X Moment (kN-m)	813830	214433	798489	207311	791671	205440
	(3.9)	(8.6)	(5.7)	(11.7)	(6.5)	(12.5)
Y Moment (kN-m)	601553	220820	590040	214911	585180	215510
	(6.3)	(9.1)	(8.1)	(11.5)	(8.8)	(11.3)
Torsion (kN-m)	69876	23010	65970	21037	64398	20913
	(13.1)	(20.9)	(18.0)	(27.7)	(19.9)	(28.1)

The results are similar for the 2% damping case except that the reductions are less significant. The TLD systems effectively increase the damping of the building, thus if the building has a higher damping, the maximum TLD effective damping contribution is reduced. Overall, there is a greater reduction in the RMS values than the peak responses.

This is beneficial for the assessment of the BMs by reducing the variance of the BMs, thus more accurately predicting the peak hourly BMs.

Following the ESWL formulation, the design loads shown in Table 3.7, Table 3.8, and Table 3.9 are obtained for the *X* direction, *Y* direction, and torsional direction, respectively. The ESWLs are shown every tenth floor for the building with and without the TLD systems installed. In addition, the two top floors are shown to clearly demonstrate the disparity in loading.

Figure 3.10 and Figure 3.11 show the reduced ESWLs along the height of the building with the TLD systems installed for the 1% and 2% damping cases, respectively. The shapes of the distributions remain approximately equal while the magnitude of the equivalent forces and torques change. The ESWLs reveal that the building with a TLD system installed will experience lower forces in the *X* and *Y* directions and significantly less torque. The significance of the reduction greatly depends on the amount of damping in the building. The TLD systems perform more effectively for buildings with lower damping ratios. Under strong wind loading, the 2% damping case is more likely to govern. Despite a less significant reduction for the 2% damping case, the reduction remains beneficial from a design perspective because the building can be designed for less loading. However, even if the reduced loading is not implemented for an optimal design, the designer has increased reassurance that the design of a building with a TLD system is conservative.

Table 3.7 The loading in the X direction for every tenth floor of the building.

Floor Height (m)	1% Damping (kN)				2% Damping (kN)			
	No TLD	TS-1	TS-2	TS-3	No TLD	TS-1	TS-2	TS-3
140.7	1536	1446	1421	1411	1431	1396	1380	1372
138.0	291	261	252	249	247	234	228	225
113.7	244	220	213	210	209	199	194	192
86.7	194	177	172	170	170	163	159	158
59.7	142	133	130	129	129	125	124	123
32.7	99	96	96	95	95	94	94	94
0.0	93	93	93	93	93	93	93	93

Table 3.8 The loading in the Y direction for every tenth floor of the building.

Floor Height (m)	1% Damping (kN)				2% Damping (kN)			
	No TLD	TS-1	TS-2	TS-3	No TLD	TS-1	TS-2	TS-3
140.7	1132	1067	1054	1050	1052	1026	1018	1015
138.0	227	189	182	180	187	170	165	163
113.7	191	161	155	154	160	146	143	141
86.7	154	132	128	127	131	121	118	117
59.7	114	102	100	99	100	95	94	93
32.7	80	75	75	74	74	72	72	72
0.0	73	72	72	72	71	71	71	71

Table 3.9 The torque for every tenth floor of the building.

Floor Height (m)	1% Damping (kN-m)				2% Damping (kN-m)			
	No TLD	TS-1	TS-2	TS-3	No TLD	TS-1	TS-2	TS-3
140.7	9669	6887	6050	5771	7283	5973	5409	5197
138.0	3180	2405	2196	2132	2513	2141	2002	1946
113.7	2721	2078	1907	1854	2159	1850	1736	1690
86.7	2154	1670	1542	1503	1724	1491	1407	1372
59.7	1528	1222	1144	1121	1248	1102	1050	1029
32.7	939	800	767	758	804	740	719	710
0.0	653	626	621	619	614	603	600	598

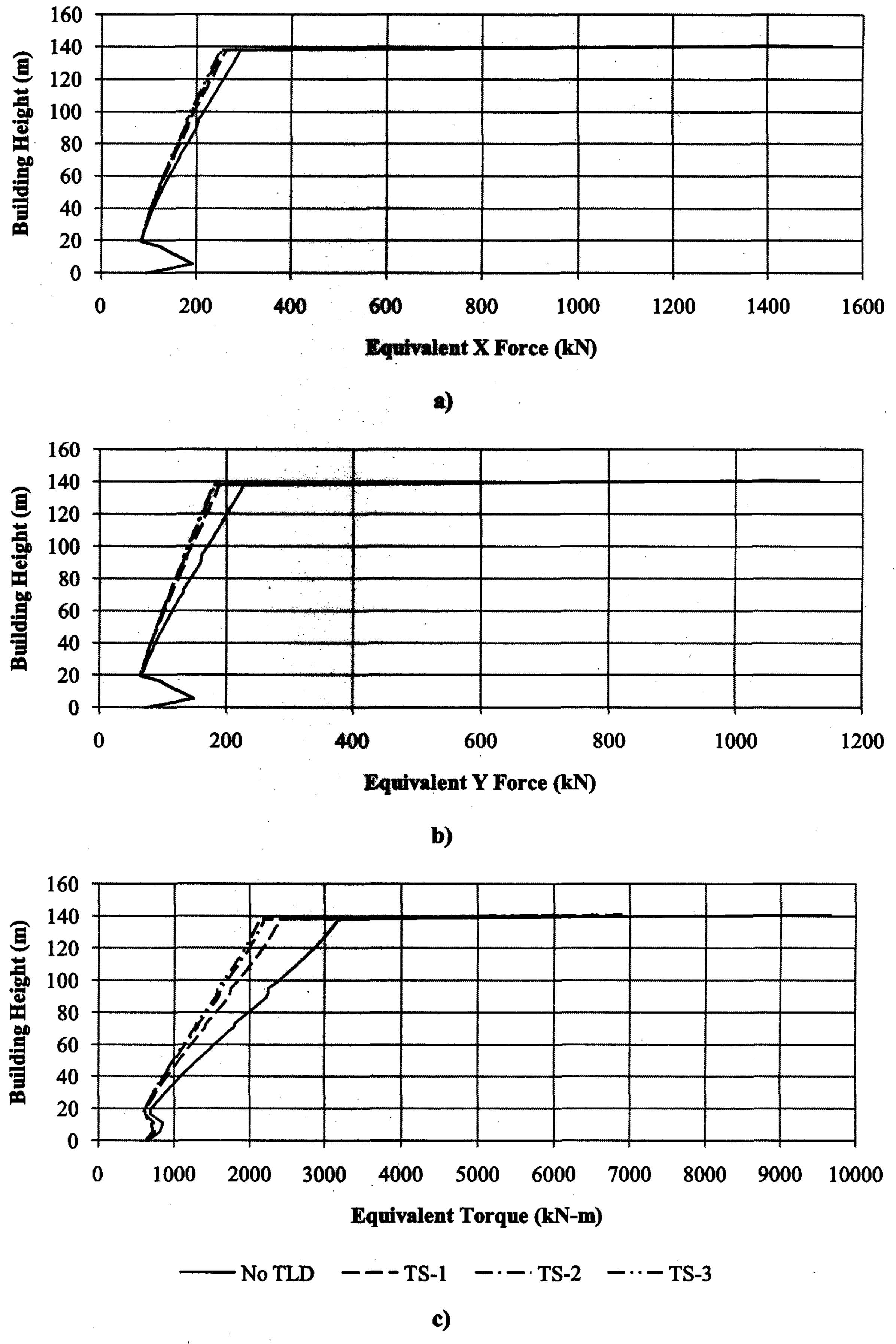

Figure 3.10 The ESWLs in the a) X direction, b) Y direction, and c) torsional direction for the 1% damped building with and without the TLD systems installed.

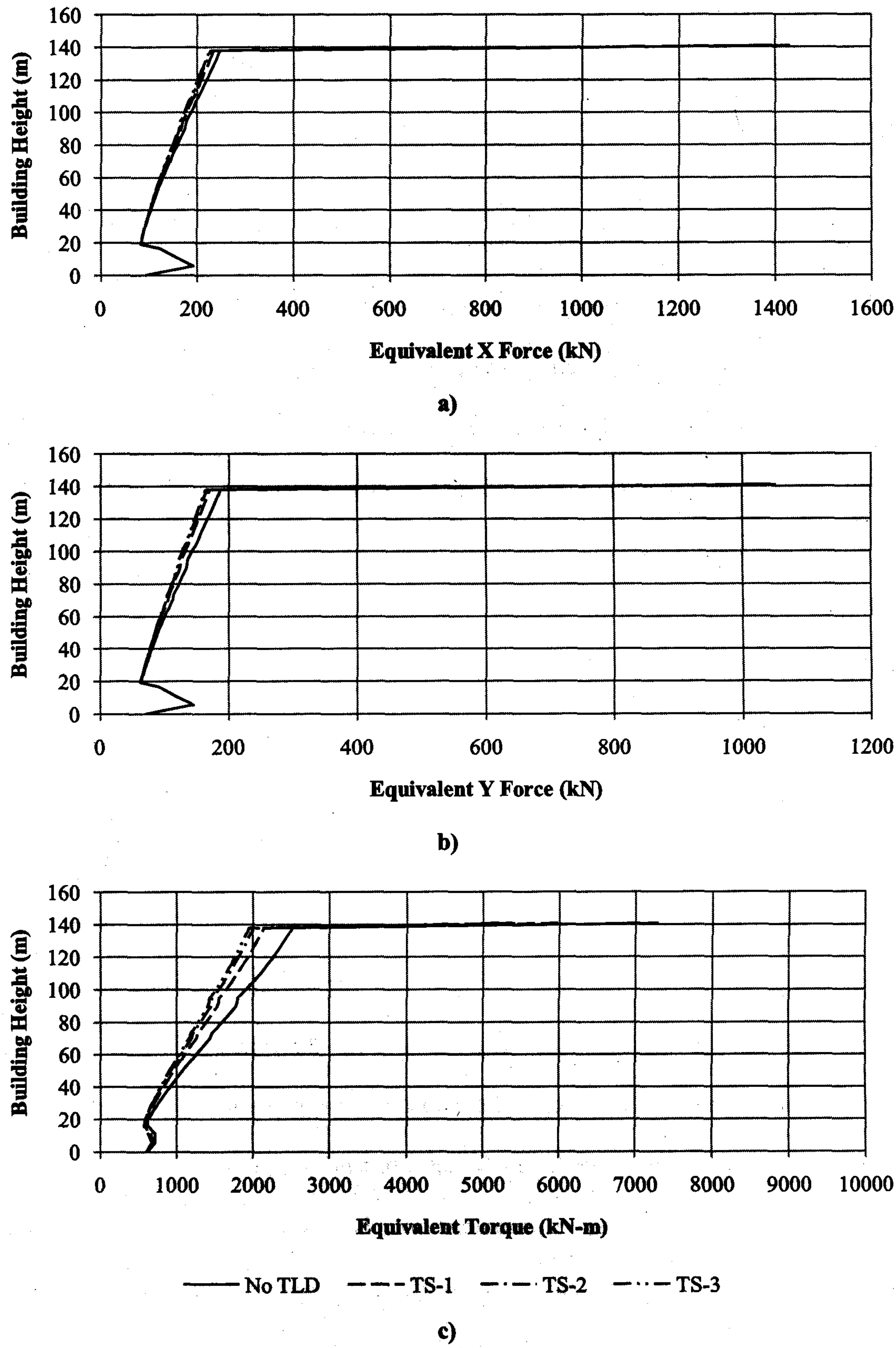

Figure 3.11 The ESWLs in the a) X direction, b) Y direction, and c) torsional direction for the 2% damped building with and without the TLD systems installed.

3.5.2 Practical Parametric Study

The concept of designing a building for reduced wind loading raises safety concerns. Most concerns derive from the need for assurance that the TLD system will perform as intended. This parametric study attempts to mitigate these concerns by evaluating the effect of two practical parameters on the effectiveness of TS-3 on the building with a 2% damping ratio. Spillage, leakage, evaporation, and poor maintenance are relevant issues that affect the water height inside the tanks; therefore, the influence of the water height on the building base responses is investigated. A second concern is in the accuracy of the evaluation of the dynamic characteristics of the building. There will likely be a deviation between the structural frequencies estimated using the finite element method and those of the as-built structure. Additionally, researchers have shown that the vibration frequencies vary with amplitude, which increases the uncertainty (Tamura and Suganuma, 1996). Therefore, the influences of the first and second mode structural frequencies on the base responses of the building are also investigated.

3.5.2.1 Effect of the Water Height

The water height, h, inside each TLD of the system is simultaneously varied by $\pm 25\%$ of the optimal water height, h_w, which is the water height corresponding to a tuning ratio of unity. Increasing the water height will simultaneously increase the TLD sloshing frequency and water mass as calculated by equations 3.12 and 3.13, respectively. Figure 3.12a and Figure 3.12b illustrate the effect of the water height on the response reduction, Ψ, of the base shears and base moments, respectively. The study reveals a moderate influence on the base responses of the structure-TLD system over the range tested. However, the influence is purely beneficial for the system because as the water

height lowers, the efficiency is increased to an approximate maximum at $h/h_w = 0.9$. In addition, the water height is unlikely to be raised, thereby almost eliminating the chance of entering the less effective range of $h/h_w > 1.0$. The water height is more readily lowered due to spillage, leakage, or evaporation. The response reduction is mainly influenced by the tuning ratio, which when reduced below unity, the TLD efficiency increases. This trend was also observed by Tait et al. (2004b).

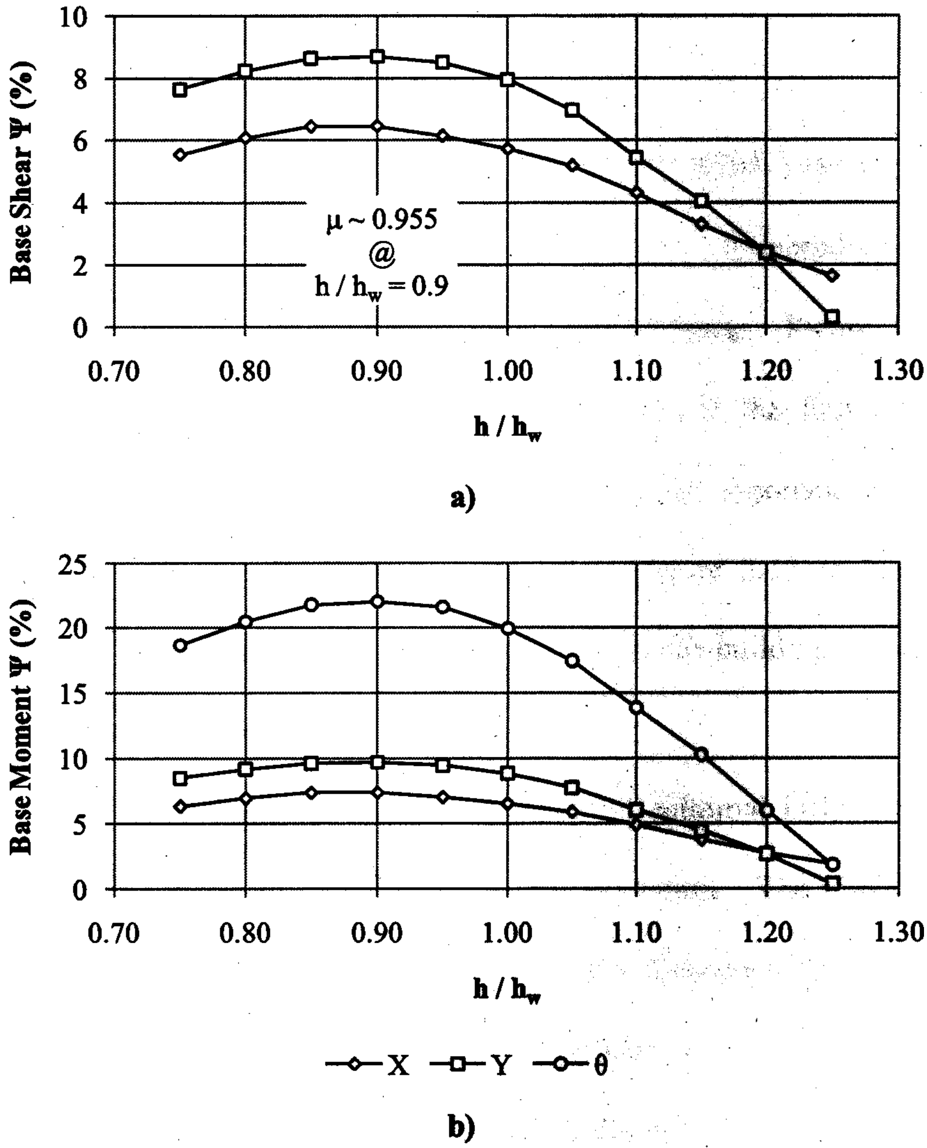

Figure 3.12 The reduction in a) base shears and b) base moments for different water depth ratios.

Provided that the TLD system has a water height corresponding to $h/h_w = 0.9$, the base shear and moment in the sway directions are relatively insensitive to a $\pm 15\%$ change in water height. Conversely, the base torque is more sensitive to changes in the water height. Nonetheless, the base responses are relatively insensitive to changes in the water height, if the TLD water height is set to an optimal value of $h/h_w = 0.9$. Therefore, as included in Figure 3.12a, an optimal TLD design should have the tanks set to a tuning ratio approximately 5% less than unity.

3.5.2.2 Effect of the Structural Frequencies

The structural frequency, f_s, is varied by $\pm 15\%$ of the modal frequency, f_j, obtained from the dynamic analysis of the building for mode j. Essentially, the structural frequency, f_s, is the uncertain as-built frequency of the building. Figure 3.13 illustrates the effect on the base shears and moments from changes in the first mode structural frequency. Changing the structural frequency affects the resonant response of the building with and without a TLD system; therefore, Figure 3.13 includes each base response separately to compare the relative effect between the building response with and without a TLD system installed.

The first mode structural frequency has a relatively minimal influence on the base shears and moments for the building without a TLD system. The slightly fluctuating relationship is due to the varying dominance of particular frequencies in the wind, thereby exciting the building with varying magnitudes. Despite this fluctuating response, the building with a TLD system is capable of regulating the base shears and moments. Each base response with TS-3 is similarly influenced. The effectiveness is increased when $f_s/f_1 > 1.0$. Conversely, when $f_s/f_1 < 1.0$, the effectiveness of the TLD system rapidly

decreases. These phenomena are directly related to the TLD tuning ratio. The TLD loses effectiveness when the tuning ratio is greater than unity. Figure 3.13e includes the optimal tuning ratio for mode 1 tanks of TS-3. Notably, the base responses in the *Y* direction are less sensitive to changes in the first mode structural frequency because the resonant contribution from the first mode is less significant than in the *X* and θ directions (see Table 3.2).

Figure 3.14 displays the results from a similar test on the second mode structural frequency. The *X* direction is insensitive to changes in the second mode structural frequency because of the insignificant resonant contribution as shown in Table 3.2. The *Y* direction, however, is susceptible to changes in the second mode structural frequency because the resonant contribution is dominant in the *Y* direction for the second mode. Once again, the TLD system is sensitive to the tuning ratio, thus resulting in the loss of efficiency for overestimations of the second mode structural frequency. However, the TLD system responds positively to underestimations, thereby indicating a moderate level of robustness. Figure 3.14e includes the optimal tuning ratio for the mode 2 tanks of TS-3.

The overall effect of varying the structural frequencies is exclusively dependent on the resulting tuning ratio of the structure-TLD system and the significance of the resonant contribution to the base responses. For TLD design purposes, if the structural frequencies are underestimated, the structure-TLD system remains effective. To avoid overestimation of the structural frequencies, in-situ dynamic tests on the as-built structure should be carried out to ensure proper tuning of the TLD system.

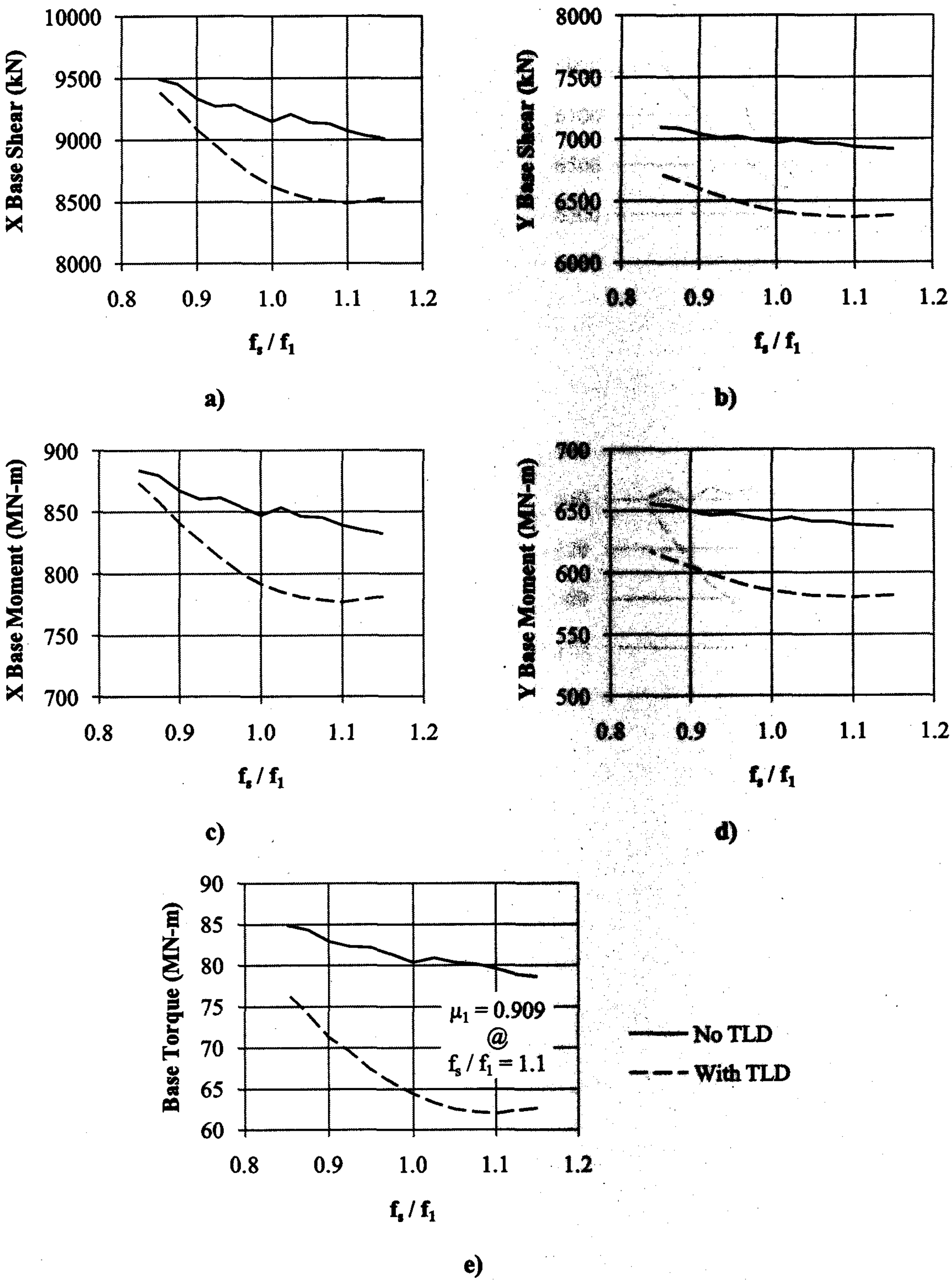

Figure 3.13 The base shears and moments for different first mode structural frequencies.

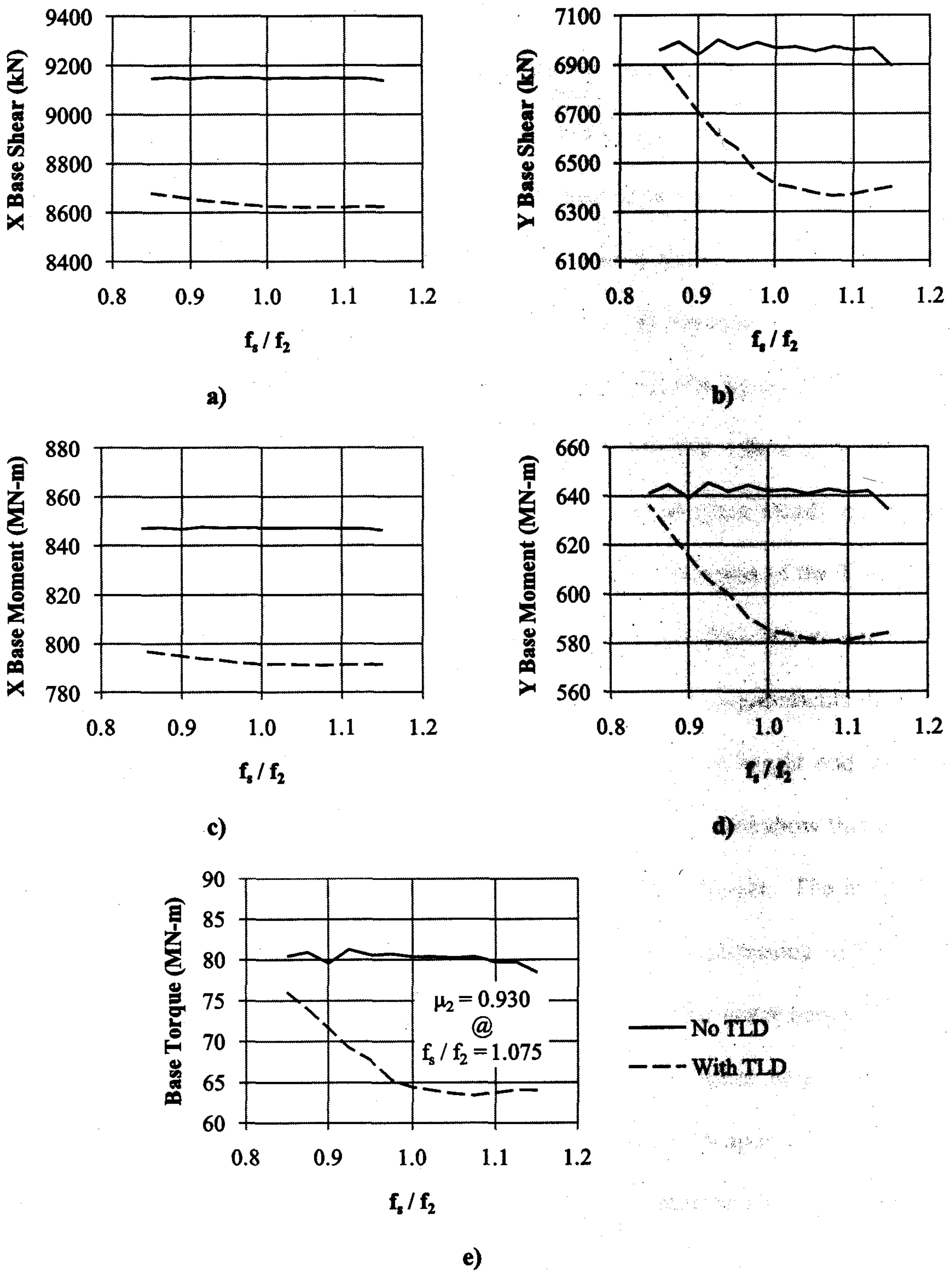

Figure 3.14 The base shears and moments for different second mode structural frequencies.

3.6 Conclusion

This chapter formulates an approach to calculate the equivalent static wind loads for a tall building with a TLD system installed. Three TLD systems with varying water mass are designed and evaluated using data from HFFB tests conducted at the BLWTL at the University of Western Ontario. The results illustrate that as the water mass is increased, the TLD systems are increasingly effective at controlling the resonant component of the wind loading. TS-3 reduced the ESWL for the torsional direction by more than 20% for the building with a 2% damping ratio. The TLD system was less effective in the lateral directions because of a more significant contribution from the quasi-steady component, which the TLD system cannot control. However, the primary function of the TLD design was to reduce the torsion; therefore, the TLD system performed effectively as intended.

Since using a TLD system to reduce the strength design requirements is a new concept, a practical parametric study was performed on the water height and structural frequencies as a means to evaluate the robustness of TS-3. The results show that the TLD system has a relatively minor sensitivity to changes in the water height. The influence is due to changes in the sloshing frequency, thereby resulting in a mistuning of the TLDs. However, the change in water height is not an issue because as the water height decreases the effectiveness is increased; in addition, an increase in the water height is unlikely because water is only readily lost through spillage, leakage, or evaporation. Therefore, based on a realistic scenario perspective, changes in the water height will exclusively benefit the structure-TLD system, over the range tested.

The study on the structural frequencies provides insight into the effect of under- and overestimations of the first and second structural frequencies on the base responses of the building with a TLD system. The study reveals that TS-3 is affected positively or

negatively, depending on whether the structural frequency is underestimated or overestimated, respectively. Furthermore, the study reveals that a tuning ratio of approximately 9% and 7% less than unity for mode 1 and mode 2 tanks, respectively, yields a more optimal TLD system design. This would provide a further reduction in the base responses. The detrimental tuning ratio (greater than unity) is simply avoided by evaluating the structural frequencies of the constructed building and fine-tuning the TLD system. Notably, the effects of the first and second structural frequencies on the base responses are dependent on the relative resonant contribution from the wind load.

Overall, the TLD systems analysed in this chapter are capable of reducing the ESWL by an amount that is great enough to influence the design of the building. Therefore, this research develops and demonstrates the concept of using a TLD system to control the design of a building that is sensitive to dynamic loading.

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CHAPTER 4

A Cost Savings Concept for Structures with Tuned Liquid Dampers

4.1 Introduction

Tuned liquid dampers are established devices in the field of dynamic vibration absorbers (DVAs). The purpose of a DVA is to control the dynamic responses of a structure. The dynamic effects contribute substantially to the total response of tall buildings, thus providing an ideal structure for the use of tuned liquid dampers (TLDs). Currently, tuned liquid dampers are designed exclusively to reduce serviceability responses, such as excessive accelerations or inter-storey drifts; however, by reducing the accelerations, the resonant shears and moments are also effectively reduced.

The concept of reducing the loads on a structure by installing TLDs is attractive from a design perspective – a reduced loading subsequently means smaller cross-sections and less reinforcement. Using less concrete, reinforcement, and steel in the design of a building ultimately translates into a cost savings. For a building that is sensitive to dynamic loading, a substantial savings is possible simply by installing a TLD system. Rahman (2007) demonstrated the ability of a TLD to reduce the rotation experienced at the beam-column connections in a reinforced concrete moment resisting frame (MRF), thereby resulting in less stress in the concrete and reinforcing steel.

Less stress corresponds to a possible reduction in the amount of reinforcing steel required for an efficient design. This chapter demonstrates the cost savings associated with a reduction in the reinforcing steel in shear walls of a high-rise building due to the installation of a multi-modal TLD system. An equivalent static wind load (ESWL) is

established for the dynamically sensitive building with and without TLDs. The ESWL is calculated using data from the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario. The shear walls are designed for both ESWL cases and the amount of steel reinforcement required for both cases is summarized. The shear wall design is based on obtaining a factor of safety (FOS) of unity for both design loadings. In addition, suggestions and techniques are recommended to ensure the TLD will perform properly and effectively under all circumstances.

4.2 Structure-TLD Assessment

The building is 50 stories tall and has a lateral load resisting system that consists of reinforced concrete shear walls and MRFs. Figure 4.1a illustrates the irregular L-shaped floor plan and shows the coordinate system. The MRFs are located along the east and south exterior faces of the building. The two main shear walls (walls 1 and 2) are located along the exterior walls where the M_x and M_y sign conventions are labelled in Figure 4.1a. There are 17 other smaller shear walls located in the core. The L-shaped floor plan causes the building to have a torsional sensitivity, in addition to weaker MRF structural system around the perimeter of the floor plan. The large eccentricity between the center of mass and center of rigidity exacerbates the sensitivity and causes strong lateral-torsional coupled modes. The first two structural modes of vibration have a significant torsional contribution.

The dynamic sensitivity causes the building to respond excessively when subjected to wind loading. High frequency force balance (HFFB) tests at the BLWTL confirmed this sensitivity. To resolve the problem, a TLD system is designed specifically to control the torsional motion of the building. As shown in Figure 4.1b, many tanks are placed around

the perimeter of the floor plan as a means to maximize the effectiveness of the TLD system at reducing the torsional motion. Two sets of tanks are designed to dissipate energy in the first mode and second mode of structural vibration because both modes contribute significantly to the torsional motion.

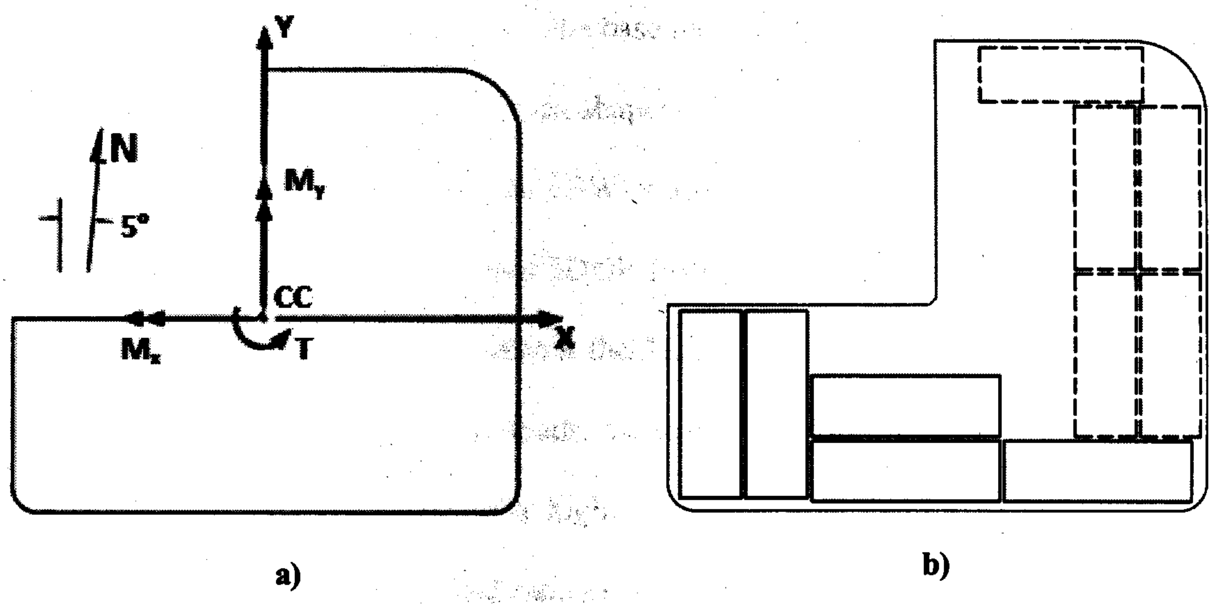


Figure 4.1 The building floor plan with a) the coordinate system and b) the TLD system arrangement for tanks tuned to mode 1 (solid line) and mode 2 (dashed line).

The TLDs dissipate the energy from wind excitation through the sloshing action inside the tanks. The sloshing action causes the TLD to behave nonlinearly with respect to the excitation amplitude. The nonlinear properties of the TLD are evaluated by using the equivalent tuned mass damper (TMD) approach (Tait et al., 2004a). This approach describes the TLD through an amplitude-dependent equivalent TMD mass, frequency, and damping ratio. These properties are implemented into a numerical program that is capable of solving a two degree-of-freedom (2DOF) problem (Tait et al., 2004b). The building generalized properties describe one DOF and the nonlinear equivalent TMD properties describe the second DOF. The 2DOF system is solved using the fourth order Runge-Kutta-Gill numerical method to produce the structural displacements, velocities,

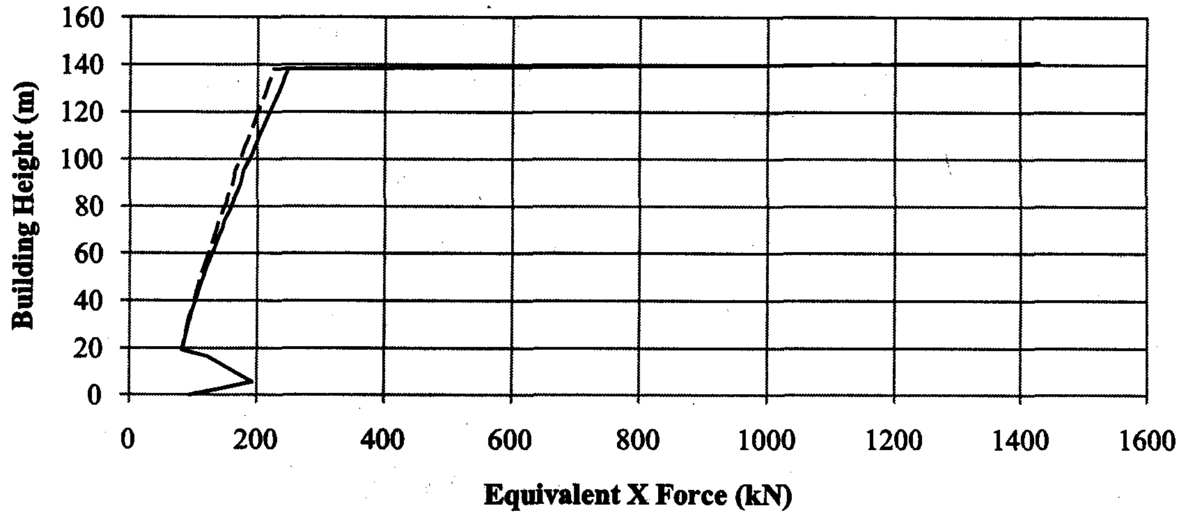
and accelerations. The accelerations are required to calculate the resonant component of the wind loading.

4.2.1 Wind Loading

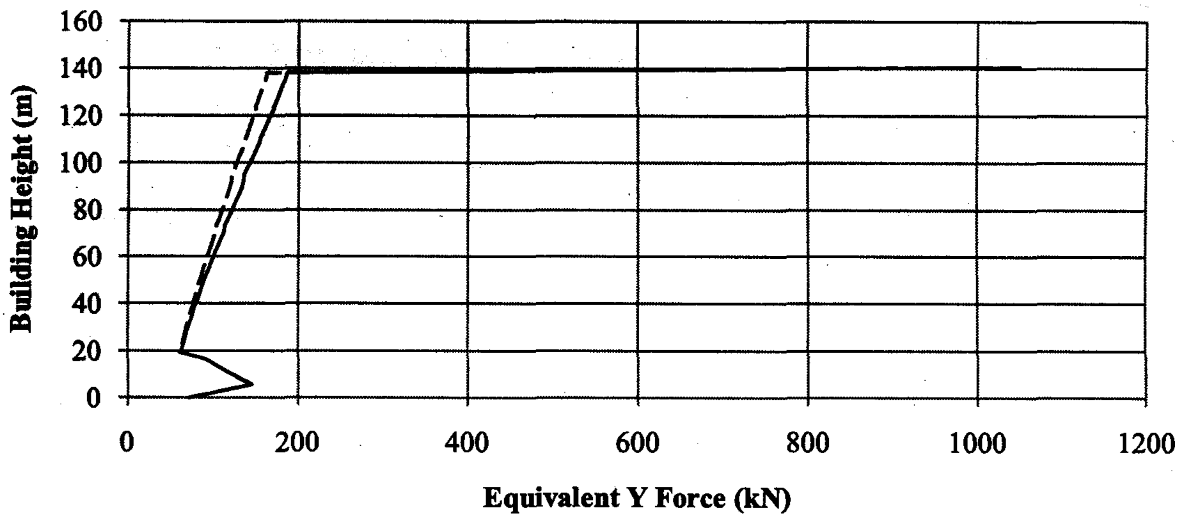
The equivalent static wind loads (ESWLs) for the building are based on the overturning moments and base torque. The base moments are distributed along the height of the building according to assumed load shapes. There is a specific ESWL for the three principle directions (X, Y, and θ). The ESWLs are solved by applying the data from the HFFB tests to the previously described 2DOF system. Figure 4.2 shows the ESWLs for the 2% damped building with and without the TLD system installed. The TLD system is capable of reducing the design wind loads, especially the torsion. The torque is reduced more significantly because there is a higher resonant contribution to the twist. The ESWLs generate the base shears and moments shown in Table 4.1, which are resisted by the shear walls and MRFs of the building. These are the base responses before applying the load combinations from Table 4.2 or the National Building Code of Canada (NBCC) load combinations for ultimate limit states.

Table 4.1 The base shears and moments caused by the ESWLs.

Base Reaction	No TLD	With TLD	Reduction (%)
X Shear (kN)	9148	8625	5.7
Y Shear (kN)	6967	6412	8.0
X Moment (kN-m)	847068	791671	6.5
Y Moment (kN-m)	641947	585180	8.8
Torsion (kN-m)	80422	64398	19.9



a)



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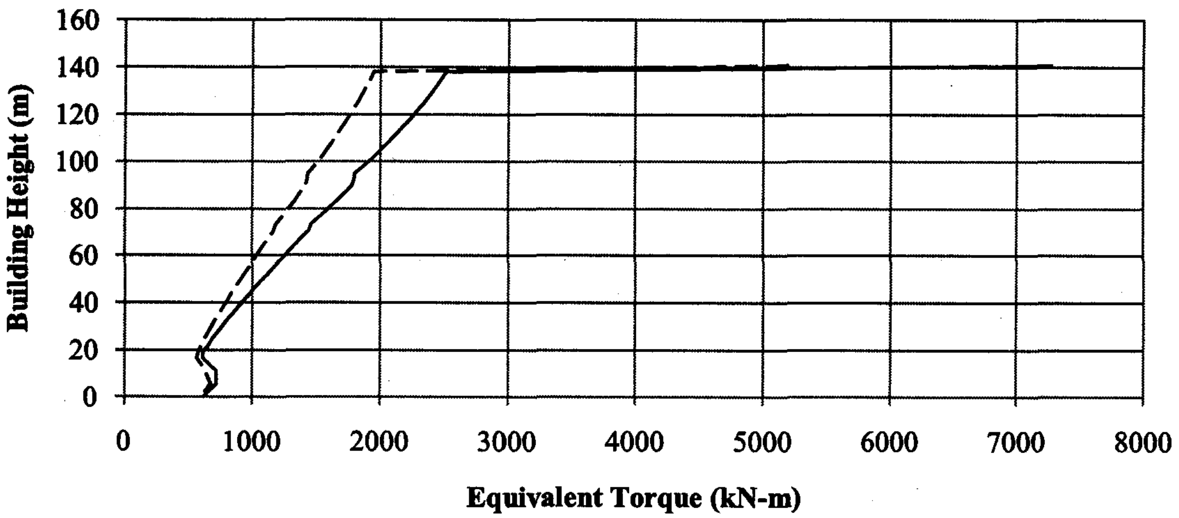

c)

Figure 4.2 The three ESWLs for the building with (dashed line) and without (solid line) the TLD system.

The ESWLs will reproduce the peak hourly base moments. However, in order to reflect the actual maximum load effects for structural design, the wind loading must be a combination of the three principle ESWLs. The combinations are determined based on the companion load concept, which states that when the peak load occurs in any one principle direction, there are additional loads in the remaining two principle directions. However, the additional loads will only be fractions of the peak loads. This accounts for the fact that the three principle ESWLs will not occur simultaneously. Accounting for positive and negative signs, a minimum of 24 cases are possible, each with a different coefficient for the three principle directions. The coefficients for each case are determined by analysing the cross-correlation between the base moments. The most critical combinations of the three directions are determined and summarized in Table 4.2. The same set of load combinations apply to the building with and without the TLD system installed because the TLD system does not affect the relative occurrence of the load effects in the three directions – it simply reduces the magnitude of the load effects.

Table 4.2 The load combinations applied to the ESWLs for the building.

Case	X	Y	T	Case	X	Y	T
1	1.0	0.45	0.9	**13**	-0.5	0.45	0.5
2	0.65	1.0	0.4	**14**	-0.5	1.0	0.4
3	0.9	0.45	1.0	**15**	-0.5	0.45	1.0
4	1.0	0.45	-0.4	**16**	-0.5	0.45	-0.4
5	0.5	1.0	-0.4	**17**	-0.5	1.0	-0.4
6	0.8	0.45	-0.6	**18**	-0.5	0.45	-0.6
7	1.0	-0.45	0.9	**19**	-0.5	-0.45	0.5
8	0.6	-0.6	0.65	**20**	-0.5	-0.6	0.55
9	0.9	-0.45	1.0	**21**	-0.5	-0.45	1.0
10	1.0	-0.45	-0.4	**22**	-0.5	-0.45	-0.4
11	0.6	-0.6	-0.4	**23**	-0.5	-0.6	-0.4
12	0.8	-0.45	-0.6	**24**	-0.5	-0.45	-0.6

4.2.2 Shear Wall Design

Combining the ESWLs with the load combinations in Table 4.2 and the NBCC, a force envelope is developed for each of the 19 shear walls. Walls 1 and 2 primarily resist lateral loading because the strong axis of each wall almost coincide with the shear center of the building. Furthermore, they resist the majority of the lateral loading because they are significantly larger than the other shear walls. The other walls carry a combination of the lateral and torsional loading.

The shear walls are designed to minimize the required materials for both with and without the TLD systems. This is achieved by designing the walls such that the FOS for the shear and moment capacity are unity. A few walls require an FOS of less than unity due to other design considerations, such as the minimum steel requirement, maximum spacing between steel requirement, and torsion capacity; however, the FOS is kept consistent for each wall between the two design cases. The 50-storey building is divided into five 10-storey sections, each of which has a unique design. The size and spacing of the horizontal, vertical, and flexural steel reinforcement is designed to achieve an efficient FOS while satisfying the NBCC design requirements. The wall thickness remains identical for each section of wall between the two design cases, thereby strictly comparing the difference in steel reinforcement. Table 4.3 summarizes the lengths of reinforcement, total mass of reinforcement, and total cost in Canadian dollars for both design cases. There is a 15.9 percent reduction in the mass of steel reinforcement required to obtain the same FOS for the building with and without a TLD system.

This is an attractive reduction from a design perspective because there is a significant cost savings of 16.9% associated with the decreased material requirement. This savings is calculated using an industry price for 15M, 20M, 25M, 30M, and 35M of $18.50,

$36.25, $55.25, $68.85, and $83.95[1] per 20 foot lengths, respectively. Based on these prices the steel reinforcement reduction will cut the total cost by $450,099. Note that this is the savings without considering the possible reduction in wasted material, the lowered shipping costs associated with the reduced mass, and the reduced time spent on arranging and tying additional reinforcement in the mesh. This cost savings analysis does not include the extra costs incurred by designing and constructing a structural support system to carry the gravity loads of the tanks. In addition, the cost of the tanks will be substantial. However, the cost savings still adds an incentive for using TLDs.

Table 4.3 The lengths, total mass, and total cost of steel reinforcement for both design cases.

	No TLD	With TLD	Reduction (%)
15M (m)	120150	119180	0.8
20M (m)	41437	35023	15.5
25M (m)	31688	17007	46.3
30M (m)	39462	33978	13.9
35M (m)	6708	6261	6.7
Total Mass (kg)	680096	572200	15.9
Total Cost ($)	2,668,743	2,218,644	16.9

4.2.3 MRF Consideration

A similar reduction in reinforcement is expected for the moment resisting frames. In fact, there is a possibility that a greater reduction may occur because the primary purpose of the MRFs is to resist the torsional loading, which has the greatest reduction (Figure 4.2c). However, the reduction in the MRF is not researched because the design is governed by seismic loading.

Rahman (2007) demonstrated the ability of a well-designed TLD system to reduce the rotation in the beam-column connections of a medium-rise MRF by over 50 percent when

[1] Prices are in Canadian dollars as quoted by Misteelco (St. Thomas, Ontario, Canada) on October 26, 2009.

subjected to various earthquake loading scenarios. The TLD system prevented the beam-column connection stresses from reaching the concrete crushing point and sequentially the steel yielding point. The reduced rotation directly correlates with a decreased moment experienced by the column, thus resulting in the possibility of reducing the reinforcement in the column. This is further evidence confirming the ability of a properly designed TLD system to reduce the strength requirements of a building.

4.3 TLD System Upgrades

Designing a structure for reduced loading simply because TLDs are installed introduces potential concerns in regards to the safety and structural integrity of the building. Maintaining a high level of system reliability is imperative to ensure the owner and tenants of the building are satisfied that the structure-TLD system will behave properly under all circumstances. To ensure the TLDs behave properly, many precautions can and should be implemented.

Maintaining an optimal tuning ratio between the sloshing frequency of the TLD and the structural frequency for the desired mode is imperative to the proper functionality of the structure-TLD system. To ensure an optimal tuning ratio, in-situ dynamic tests should be performed on the building to acquire the true structural frequencies. This is possible by recording the motions of the building over a specified time and utilising the random decrement technique to determine the structural frequencies and damping ratios (Asmussen, 1997). Using the newly established structural frequencies, the TLDs can be fine-tuned by varying the water height (Tait et al., 2004b).

Secondly, maintaining an optimal water height inside the tanks is critical to the proper functionality of the structure-TLD system. The water height may vary due to spillage,

leakage, evaporation, or poor maintenance. This will result in a shift of the water mass and sloshing frequency, thereby potentially reducing the effectiveness of the TLD. A monitoring system should be installed to measure the water height continuously and alert maintenance crews when the water height deviates by a predetermined critical amount. Furthermore, an automated system could be installed to add or remove water as needed.

Rahman (2007) and Fujino and Sun (1993) verified methods to mitigate the detrimental effects of mistuning by means of setting many TLDs to slightly different tuning ratios. In other words, each tank will have a different sloshing frequency such that the TLD system will capture a range of tuning ratios around the optimal value (i.e. $\pm 5\%$). In fact, this multiple tuned liquid damper (MTLD) system offers a considerably greater reduction in the resonant responses than offered by a single TLD system (Rahman, 2007). This is possible by capturing a larger range of frequencies in the wind or earthquake loading. This technique should always be implemented when the TLD system consists of more than one tank.

These three techniques will promote a robust and effective TLD system, thus adding considerable reassurance that the TLD system will perform as designed. More specifically, the TLD system will effectively reduce the ESWLs under all circumstances, thus providing a safe opportunity for a more optimal strength design.

4.4 Conclusion

This chapter demonstrates that a properly designed TLD system can significantly affect the strength design of a dynamically sensitive building. The building uses a combination of reinforced concrete shear walls and moment resisting frames to resist strongly coupled lateral-torsional motion. To reduce the dynamic sensitivity, a multi-

modal tuned liquid damper system is designed for the first and second modes of structural vibration. This considerably reduces the torsional response, which is reflected in the ESWLs for the building with the TLD system installed. Using the ESWLs with the properly calibrated load combination factors, the shear walls are designed for a reduced wind loading. The results confirm that the structure-TLD system requires less steel reinforcement to achieve the same FOS as the building without a TLD system installed. The reduced reinforcement amounts to a substantial cost savings of $450,099. This cost savings will likely outweigh the cost of manufacturing and installing the TLDs.

As a means to ensure that the TLD system performs properly, three precautions should be taken into consideration. First, in-situ testing should be conducted to determine the true dynamic characteristics of the building, thereby allowing proper tuning of the TLDs. Secondly, a water height monitoring system should be installed to ensure that the water height remains within an acceptable range. Finally, for a TLD system with more than one tank, each tank should be tuned such that the system captures a range of tuning ratios around the optimal value. These techniques will safely allow strength design using reduced wind loading without affecting the structural integrity of the building.

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CHAPTER 5

Conclusion

5.1 Summary

The focus of the research in this thesis is to investigate the effectiveness of tuned liquid dampers (TLDs) in reducing wind-induced torsional responses of a tall building. Previous studies have only investigated the effectiveness of TLDs in reducing lateral motions. The building chosen for this study exhibits highly coupled lateral-torsional motion, which is confirmed through modal analysis and wind tunnel testing. The tests were conducted at the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario using the high frequency force balance (HFFB) technique. The wind tunnel test results demonstrate that the building exceeds recommended serviceability limits for accelerations and torsional velocity.

To mitigate this problem, the building requires an auxiliary damping device to reduce the excessive responses to a level below the perception threshold of the occupants. The ideal device for this building is TLDs because the peak hourly acceleration responses correspond to the range in which TLDs are most effective (Tait et al., 2008b). Therefore, three unique multi-modal TLD systems are designed specifically to damp torsion. The systems are labelled TS-1, TS-2, and TS-3 in order of increasing water mass. The TLD system designs consist of two sets of tanks – a set tuned to each of the dominant torsion modes of the building (mode 1 and mode 2). In addition, the designs utilise an innovative technique to maximize the effectiveness of each tank in the system by placing the tanks around the perimeter of the floor plan. This introduces a large eccentricity between the

tank and the center of mass of the building, which allows the TLD to experience a greater displacement from the combined lateral-torsional motion. Since the behaviour of TLDs is nonlinear, a TLD performs more effectively when subjected to greater displacements (Reed et al., 1998). Furthermore, each tank in the TLD system has a pair of slat screens to increase their effectiveness by providing more control of the sloshing motion, mitigating the potential of wave breaking, and increasing the inherent damping.

For analysis purposes, the complex nonlinear behaviour of a TLD requires a simplified representation without compromising accuracy. Tait et al. (2004a) developed a numerical model that is capable of representing a TLD by an equivalent amplitude-dependent tuned mass damper (TMD). The numerical model is used to solve an equivalent amplitude-dependent TMD mass, frequency, and damping ratio for each mode of the three TLD systems. Since there are mulitple tanks in the system, a technique is developed to represent the similar tanks tuned to each mode by a single lumped TLD for each mode. This is required because the eccentric placement of the tanks results in each tank experiencing a different displacement; therefore, leading to slightly different equivalent TMD properties. This study developes the amplitude modification factor (AMF), which is a normalized weighted-average of the generalized water mass of each tank multiplied by the mode shape at the location of the tank. The AMF is applied to the generalized displacement of the primary structure to calculate the average amplitude experienced by the TLD system, thereby resulting in average equivalent TMD properties. A parametric study on the AMF demonstrates that this technique is acceptable for the structure-TLD configuration in this research. Therefore, the equivalent TMD properties define the mass, stiffness, and damping ratio of the auxiliary system attached to the primary structure.

The primary structure is defined by a single degree-of-freedom (SDOF) system with the generalized mass, stiffness, and damping ratio of the building. This defines the building without a TLD system installed. The SDOF system with the auxiliary equivalent TMD system attached defines the structure-TLD system, which is a two degree-of-freedom (2DOF) system. The SDOF and 2DOF systems are mode-specific. This study accounts for the first three modes of vibration. In addition, the generalized wind force (GWF) is mode-specific and is applied to the primary structure DOF for both systems. Using the HFFB test data, the GWF is formulated to accommodate the nonlinear 3D mode shapes exhibited by the building.

The analysis of the SDOF and 2DOF systems uses the fourth order Runge-Kutta-Gill iterative numerical method, which accounts for the equivalent nonlinear TMD. The systems are solved for the first three modes and combined through addition to form a time series for each structural response. To predict the peak hourly response from the time series accurately, the Lieblein BLUE technique is implemented. This technique fits extreme-value data to a Type I extreme-value probability distribution. The distribution framework provides for an accurate and controlled prediction of the peak hourly responses of the building. Furthermore, a site-specific wind climate model is applied to the peak hourly responses to account for the directionality of the wind loading.

5.1.1 Serviceability Limits

To check the serviceability limits, the peak hourly displacement, velocity, and acceleration responses from the structure-TLD system are analysed at the top floor. A 10-year wind speed is used in the analysis – the return period for serviceability limit responses. The results illustrate that the three TLD systems are capable of significantly

reducing the deflections, torsional velocity, and accelerations. Furthermore, these responses are reduced to levels below the acceptable limits for the occupants when TS-3 is installed. By comparing TS-1, TS-2, and TS-3, the effectiveness of a TLD system evidently increases as the water mass increases. However, there is an apparent upper limit to the amount of water mass, which is specific to a particular response. For instance, the reduced translational responses are approximately equal for TS-2 and TS-3 but TS-3 displays a greater torsional reduction than TS-2. The parametric study on the AMF also confirms this limitation. Overall, the torsional responses are reduced most significantly – achieving a 47% reduction for the building with a 1% damping ratio. Moreover, if the optimal water height from the parametric study is implemented, the structure-TLD system experiences a 56% reduction in torsional responses. This study proves that the innovative multi-modal TLD system design is capable of effectively reducing the serviceability limit responses of the lateral-torsional coupled building.

5.1.2 Equivalent Static Wind Loads

The wind acting on a building can be described by three response-specific equivalent static wind loads (ESWLs). There is an ESWL corresponding to the *X* direction, *Y* direction, and torsional direction. Furthermore, each ESWL has a mean, background, and resonant component. Each ESWL is formulated to reproduce a specific response such that when the response-specific ESWL is applied statically to the building, the obtained response exactly matches the predicted response induced by the fluctuating wind. All other obtained responses from the statically applied ESWL are simply good estimates. The widely accepted response for the ESWL is the base moment (BM) (Zhou and Kareem, 2001). In addition, using the BM as the response is ideally suited for HFFB test

data because the HFFB tests directly record the BMs.

Using the HFFB test data and the structure-TLD numerical model, the mean BM and the peak hourly background and resonant BMs are calculated. These three components are distributed along the height of the building according to assumed load shapes and are combined using addition and the SRSS method to produce the total ESWLs. The results show that the resonant component for the building without a TLD system contributes up to 60% to the total ESWLs. This is an important measure because the TLD system is limited to controlling the resonant component. However, the resonant component for this building is significant, thus presenting an ideal opportunity to investigate the effectiveness of the three TLD systems in reducing the total ESWLs.

A 50-year wind speed is used in the analysis of the ESWLs – the return period for strength design. In addition, the structure-TLD system is evaluated using a 1% and 2% damping ratio for the building because of the associated uncertainty. Similar to the serviceability limit results, the ESWL results illustrate that the TLD system effectiveness increases as the water mass increases; however, there is no apparent response-specific limitation for the water mass since TS-3 performs the best in reducing both the lateral and torsional motions. For the 1% damping ratio, TS-3 reduced the translational base shears and moments by almost 16% and the torque by 30%. These reductions are not as substantial as for the serviceability limit responses because the ESWL includes an uncontrollable quasi-steady component. For the 2% damping ratio, TS-3 reduced the translational base shears and moments by almost 9% and the torque by 20%. The 2% damping ratio is likely more representative for a reinforced concrete building subjected to strong wind loading. Despite the less significant reduction at the 2% damping level, the building experiences substantially reduced wind loading. To the best of the author's

knowledge, this is the first demonstration of a TLD system in reducing the design wind loading of a structure.

5.1.3 Practical Parametric Study

A practical parametric study is conducted to determine the effectiveness and robustness of the TS-3 system in reducing the serviceability limit responses and ESWLs. The term practical refers to a parameter that is readily variable and immediately influences the design of a TLD system. The two practical parameters investigated in this study are the water height inside the tanks and the structural frequencies. Both parameters affect the tuning ratio between the TLD and the structure. The general conclusion from the study is that a proper tuning ratio is imperative for an effective structure-TLD system. Furthermore, the system remains effective for a range of tuning ratios between 0.86 and 1.02. This substantial range demonstrates the robustness of a well-designed TLD system.

Additionally, the study on the water height demonstrates that changes in the water height are strictly beneficial for the structure-TLD system because an increase in the water height is unlikely. These observations are based on a water height that provides a structure-TLD tuning ratio of unity, which is a commonly used ratio for TLD design. When the water height increases, the effectiveness of the TLD in reducing the responses is compromised. In fact, for significant increases in the water height, the TLD system exacerbates the structural responses. Conversely, a decrease in the water height increases the response reduction. A decrease in the water height is far more likely due to potential leakage, spillage, evaporation, or poor maintenance; therefore, the TLD system remains effective and robust despite any realistic change in the water height.

The uncertainty associated with modal analysis estimates of the structural frequencies provides the impetus for the study on over- and underestimations of these structural frequencies. The study demonstrates that overestimating the structural frequency for first and second modes severely decreases the effectiveness of TS-3. Conversely, underestimating the structural frequencies has a minimal effect on the performance of TS-3. In addition, TS-3 is capable of regulating the structural responses despite the varying effect of the frequency content in the wind. This exemplifies the robustness of a properly designed TLD system. Notably, TS-3 will perform optimally at a tuning ratio of 0.93 and 0.95 for serviceability limit responses and strength design, respectively.

5.1.4 Strength Design

The potential for a TLD system to reduce the structural design requirements is investigated by designing the shear walls of the lateral-torsional coupled building, with and without TS-3 installed. The objective is to compare efficient designs; therefore, both designs aim to achieve factors of safety (FOS) of unity for each shear wall. The wall sizes are kept constant while the steel reinforcement changes between the two design cases. The case with TS-3 installed produced a 15.9 percent reduction in the mass of steel reinforcement required to produce the same FOS as the building without a TLD system installed. Based on industry prices for steel reinforcement, implementing TS-3 provided a material cost savings of $450,099 – a 16.9% reduction. This is an example of the cost savings potential inherent to a well-designed TLD system.

As a means to ensure the structural integrity and safety of the building, three techniques should be implemented. First, in-situ testing should be conducted on the constructed building as a means to provide an accurate assessment of the structural

frequencies, thereby allowing for optimal tuning of the TLD system. Secondly, a water height monitoring system should be installed as a means to ensure that the water height remains within an acceptable range. Finally, if the TLD system consists of more than one tank, each tank should be tuned to a slightly different sloshing frequency around the optimal tuning ratio. This captures a larger frequency bandwidth and increases the effectiveness and robustness of the TLD system (Fujino and Sun, 1993).

5.2 Future Studies

The next logical step is to perform measurements on a full-scale structure-TLD system to validate the TLD design approach and the structure-TLD numerical model used in the analysis. This research would also provide valuable insight into the reliability of the TLD system for strength design considerations. Furthermore, the development of a three-dimensional structure-TLD analysis program would be beneficial for predicting the responses of a building simply and accurately for TLD systems with complex tank configurations. This would eliminate the need for the AMF, which is introduced in this research. Finally, the reduced ESWL and cost savings concepts should be researched for other structural applications, including steel buildings, composite steel and concrete buildings, and long-span bridges.

5.3 References

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