Displacement-Based Design of Reinforced Concrete Moment Resisting Frame
Incorporating Cross Laminated Timber Infill and Metallic Damper Connectors

by

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Abstract

This thesis discusses the development of a new innovative reinforced concrete hybrid structure. The hybrid structure consists of reinforced concrete frame incorporated with Cross Laminated Timber (CLT) and metallic damper connections. The seismic design of this proposed system was carried out with the displacement-based design framework and the design was successfully verified.

First, this study focused to numerically model the conventional metallic (steel slit) damper and validated with the experimental result using the Abaqus finite element program. Then, to minimize the drawbacks of the conventional damper specimen, a parametric study has been carried out by changing the shape parameters of the damper using the factorial design of experiments. The purpose of conducting a parametric study is to find the appropriate configuration of the damper which can perform well with the proposed hybrid system. Further, the importance of the shape parameter and their interactions in the final response was studied using the response surface method. Secondly, the proposed hybrid system with the metallic damper connection was modeled in Extended Three Dimensional Analysis of Building Systems (ETABS) and then the overall behavior of the system was investigated.

In addition, a direct displacement-based design framework was developed for the seismic design of this proposed system. To verify the proposed framework, a 2D six storey hybrid structure was modeled using ETABS. Then, a nonlinear time history analysis was conducted for the modeled structure using 50 set of ground motions to evaluate its performance. The results indicate that the proposed design framework is effective in controlling the displacement of the hybrid system under seismic excitation.
Lay Summary

Timber based hybrid structures are becoming popular in Canada. These structures are constructed by combining mass timber called Cross Laminated Timber (CLT) and with reinforced concrete or steel structural systems. During the seismic event, these hybrid structures performance can be increased by using proper energy dissipative connections, for example, dampers. By considering the above, this thesis developed a new reinforced concrete hybrid structure by incorporating CLT and metallic dampers.
Preface

This thesis work is based on the research work conducted in the School of Engineering at The University of British Columbia Okanagan Campus under the supervision of Dr. Solomon Tesfamariam. All the literature review, mathematical calculations, and simulations of this thesis are carried out by the author. A list of publication at The University of British Columbia is listed as follows.

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Dedication

To my dad
Chapter 1: Introduction

Canada is enriched with a vast availability of forest resources. This made the practising engineers, architects and builders to choose wood as a primary source of construction material. From the 1850s to the 1940s, Canada had a strong record in the construction of several historical tall wood buildings mainly in Toronto and Vancouver. The buildings were built with un-reinforced brick with mortar in exterior walls and heavy timber beams inside and they are commonly called “Brick and Beam buildings” (B and B). As these buildings were constructed without consideration of seismic and fire safety regulations, the building codes in British Columbia imposed a strict height limitation of six storeys to ensure fire safety. In recent decades, the advancement of timber construction has produced a new mass timber product called “Cross Laminated Timber” (CLT). Unlike ordinary light frame wood structures, CLT has a unique contribution in resisting fire and increasing the building height (Green and Karsh, 2012; SOM, 2013).

The CLT is prefabricated from the solid engineering wood product called “lumber”. During the manufacturing process, these lumbers are arranged in orthogonal layers as shown in Figure 1. Each layer is bonded together by using strong adhesives. After the layers reach the required thickness, the layers are pressed together to form a mass timber called “CLT”. The thickness of panels ranges from 99 to 309 mm, and it differs from manufacturer to manufacturer. The use of CLT in the structure has multiple advantages such as it reduces the carbon footprint, reduces the installation time and it is five times lighter than the concrete wall and increases the aesthetic appearance (Structurlam, 2016).
This mass timber has been used as floor or wall panels for the residential and commercial building projects because of its high strength and weight ratio. Due to these advantages, the CLT has received better attention among the construction industries and researchers. One of the notable tallest contemporary wood buildings in the world is the Wood Innovation and Design Centre (WIDC) located in Prince George, British Columbia, constructed in 2014 using CLT. This structure is 29.5 m tall, consists of glulam beam and column and with the CLT core. Several studies have carried out experimental investigation to understand the behaviour of CLT under lateral loading. Ceccotti et al. (2006) carried out extensive research about the CLT under the SOFIE – a cooperative research project in Trento province, Italy. The study indicated that CLT is a very stiff and strong material. However, the energy dissipation and the ductility of the system is mainly contributed by the connections. The study conducted reverse cyclic loading test for the wall panels with hold down and steel angle anchors. They found that the stress concentration was observed by the local failure of the connectors.
Further, to gain more acceptance of the CLT product in North America, researchers (Popovski and Karacabeyli, 2012; Karacabeyli and Brad, 2013) at FPInnovation, Canada conducted a series of experimental test for the CLT wall panel with various aspect configuration and connection details. The test results indicated that the adequate seismic performance of the CLT was achieved under loading, but the connections are the critical regions for the failure. Schneider (2015) conducted the experimental study for the bracket connection by placing it parallel and perpendicular to the CLT as shown in Figure 1.2 (a) and (b), respectively. The bracket connections are connected to the CLT by using nails. The author performed a series of monotonic and reverse cyclic test to understand the behaviour of these connections. From the test results, nails pull-out failure was noted. To avoid those nail failures, Schneider, (2015) proposed a new steel tube connector for the mass timber panels, particularly for CLT. This study addressed about the feasibility of steel tube connectors in terms of damages, stiffness and ductility.

![Image](image1.png)

Figure 1.2 (a) L-Bracket connection tested with CLT in parallel and (b) perpendicular direction ([Schneider et al. 2014] by permission from publisher)
1.1 Timber based hybrid structural system

Timber has the tremendous advantage of reducing carbon foot print, increasing aesthetic appearance in high rise buildings and providing lighter structure. From the seismic point of view, timber exhibits a brittle nature when it is subjected to shear loading due to its non-ductile behavior. The ductile behaviour can be enhanced by providing the proper steel connection with the timber material. This led the researchers to think about the timber based hybrid structural systems (Green and Karsh, 2012; SOM, 2013; Fairhurst, Zhang, and Tannert, 2014; Tesfamariam and Stiemer, 2014). These hybrid systems can be classified as component level and system level hybridization. When two or different materials such as steel or concrete are combined with wood to act as a single structural member, then it is termed as component level hybridization. This includes hybrid beams (e.g., Flitched beams), hybrid columns or hybrid slabs. In some cases, the component level hybridization is carried out by providing the innovative unbonded post tensioned connections in the timber elements to limit the residual deformation of the structure (Hybrid joints). System level hybridization refers to different structural elements such as steel, concrete or timber combined at the global level to perform as a single structural unit (e.g., steel frame-timber, concrete frame-timber).

Finding the Forest Through Trees (FFTT) is the first hybrid concept developed by Micheal Green and Eric Karsh in 2008 (Green and Karsh, 2012). This system is proposed for buildings as high as 30 storeys. Basically, FFTT was to develop a timber-steel hybrid system. In this system, mass timber panels were used for the development of vertical members, shear walls and floor slabs. The ductility of the system is ensured by providing steel beam connections with the mass timber panels. Hence, in the developed mechanism, the beam acts like a sacrificial component to protect the integrity of the timber elements.
Skidmore, Owings, and Merill (SOM, 2013) proposed a conceptual mass timber-concrete hybrid system. This concept was to develop a mass timber as the primary structural system with supplementary reinforced-concrete at the joints to ensure the ductility of the system. The structure is aimed for up to 42-storeys (120 m) in height. This report highlights that the successful implementation of this concept can reduce the carbon foot-print up to 75%.

In the past decades, several researchers (Xiong and Jia, 2008; Khorasani, 2011; Dickof, 2013; Tesfamariam and Stiemer, 2014; Bezabeh, 2014; Schneider, 2015; Yazdinezhad, 2016; Isoda and Tesfamariam, 2016; Isoda et al., 2017) have contributed to the development of system level hybridization (Timber-steel and timber-concrete). Some of the literatures are discussed below.

### 1.1.1 Timber-steel hybrid structural system

The research conducted at UBC Okanagan (Dickof 2013; Tesfamariam et al. 2014; Tesfamariam et al. 2015; Bezabeh 2014; Goertz 2016) have developed a new hybrid structure by incorporating CLT as an infill in steel moment resisting frames (Figure 1.3). This hybrid system was developed by choosing L-shaped bracket connection to provide energy dissipation capacity of the system. Schneider (2015) conducted a series of experimental test on the bracket connections with the CLT. The bracket connection test results were used in the numerical model development of CLT infilled steel moment resisting frame.

![Figure 1.3 CLT infilled Steel Moment Resisting Frame (3, 6 and 9 storey)](image)
Dickof (2013) carried out an extensive research on CLT infilled steel moment resisting frame. The CLT infill was connected to the steel frame using the steel bracket connections. First, this research conducted a parametric study on a single bay single storey model to understand the effect of panel parameters such as panel thickness, panel crushing strength and confinement gap. Based on the parametric study, the author found that the panel thickness and strength did not have much influence on increasing the strength of the system. Further, Dickof (2013) indicated the system yield point for each frame type, infill configuration and the storey height with regards to the bare frame. This research has contributed in finding the force reduction factors for the seismic design of this hybrid system. Finally, the author recommended the ductility and overstrength value for the system to be 2.5 and 1.25, respectively (Dickof, Stiemer, Bezabeh, and Tesfamariam, 2014).

Bezabeh (2014) studied the lateral behavior of CLT infilled steel moment resisting frame using the response surface method, the genetic algorithm and artificial intelligence techniques. First, the response surface method was used to develop an equation for maximum interstorey as a function of residual interstorey drift and modeling parameters for the hybrid system. The model was subjected to twenty earthquake ground motions to find the maximum interstorey and residual interstorey drift with different modeling variables. Then, the maximum interstorey drift equation was represented in terms of residual interstorey drift and modeling parameters were statistically validated. From the results, it is noted that the addition of infill decreases the maximum interstorey drift. Further, the study extended to find the optimum modeling variables for the hybrid system by considering the maximum interstorey drift and residual interstorey drift as an objective function using the genetic algorithm. The author determined these optimum
modeling variables such as bracket use as an input in their full-scale hybrid system design and modeling.

Recently, the research conducted by Goertz (2016) have developed a timber-steel core wall system. This hybrid system consists of CLT panel with steel plates and ductile steel connectors. The CLT wall was connected at the intersection of each floor by using T-stub connections. The T-Stub connection was accounted to provide the necessary energy dissipation and ductility for the proposed system.

1.1.2 Timber-concrete hybrid structural system

In recent years, the research towards timber-concrete hybrid structural system has been receiving much attention. Some researchers have focused their work on timber-concrete hybrid structures (Xiong and Jia, 2008; Yazdinezhad, 2016; Isoda and Tesfamariam, 2016; Isoda et al., 2017). Xiong and Jia (2008) carried out the shake table test for the 3-storey full-scale wood-concrete hybrid building which consists of a concrete frame at the bottom storey and a wood-frame construction at the top two storeys. The author studied the dynamic characteristics (natural frequencies and damping ratios) for the five hybrid specimens with different configurations and different stiffness ratios between the concrete frame and the wood frame. The natural frequencies and damping ratio of the five specimens lies in between 2.5 – 5 Hz and 4 – 5 %, respectively. They concluded that the seismic response is greater for the specimen with smaller stiffness ratios.

Yazdinezhad (2016) investigated the application of CLT as an infill panel for the 4-storey reinforced concrete structure in reducing the torsional effects. The study finds that the hybrid structure was effective in reducing the building base shear to 40% than the original structure.
Also, the torsional sensitivity index for the hybrid structure lies well below the code specified limit (1.7).

Isoda et al. (2017) developed a timber-concrete hybrid system and performed a shake table test under increasing seismic intensity. This hybrid system consists of a reinforced concrete shear wall connected to the glulam column, beam and plywood flooring. The plywood floor is supported by the glulam timber beams. The authors investigated three engineering demand parameters such as maximum interstorey drift, residual drift and maximum acceleration for the three tested specimens. The specimens 1 and 2 consist of one storey timber frame – RC core wall with plywood flooring and concrete topping, respectively. Specimen 3 was two storey timber frame – RC core wall. During testing, the response of the building was monitored by using 200 sensors which includes all accelerometers, strain gauges, displacement transducers and load cells. Before testing, the natural period and damping ratio were determined by subjecting the structure under triangular load. The shake table test was conducted for the three specimens with one artificial earthquake record of increasing intensities and additionally S2, S3 was subjected to the Kobe earthquake record. From the response of the parameters, the researchers found that the Kobe earthquake induced severe pounding damages on the RC core wall – timber frame connection. Further, they concluded the seismic demand on the connection can be reduced by providing the expansion joints between the connection and RC core wall.

Isoda and Tesfamariam (2016) focused their studies to understand the connection behaviour of the above developed timber- RC core wall hybrid building. The connection has an important role in the energy dissipation by undergoing inelastic deformation of materials. The authors conducted experimental study to understand three connection regions, timber beam-column connection, RC wall – timber beam connection and timber column-base connection. The results
obtained for the connection were calibrated using the *pinching-4 material* model in OpenSees. In addition, the calibrated material model parameters for the connection was used to model a portal frame. The portal frame with the calibrated connection were subjected to monotonic and reverse cyclic loading. The authors found that the analytical model was capable to capture the initial stiffness of the experimental results. Further, this research contributed by providing the calibrated material parameters for the connections, which can be used in large scale numerical modeling of this hybrid system.

1.2 Motivation

The above studies conducted by the researchers on the timber based hybrid system is motivating to develop the CLT-RC hybrid system with metallic damper connectors (Figure 1.4). This hybrid system consists of CLT as a partially infilled in the reinforced concrete system with the metallic damper connectors. There are different metallic dampers which can be installed in the structure to improve its seismic performance. In this work, a steel slit damper is considered as a damper connection between the CLT and the beam. The proposed system is avoiding the infill material contact with the column to avoid the premature column failure. Further, in the conventional seismic design (force based) procedure, the lateral forces are calculated by using reduction factors such as over strength and ductility. This proposed system is a new one and the use of force based design with the unknown force reduction factors (Rd and Ro) is not possible. To overcome the design issues, a direct displacement-based design developed by Priestley (2000) has been undertaken for the seismic design of this hybrid system. A detailed review about direct displacement-based design is provided in Chapter 2.
1.3 Application for Retrofitting

In the conventional reinforced concrete retrofitting practice, the strengthening of the reinforced concrete structure is focused to increase the strength, stiffness and ductility (Thermou & Elnashai 2006) of the structure by using system level or element level retrofitting techniques. In the system level, the strengthening is provided by adding shear wall, steel bracing and by external precast panels; whereas element level strengthening includes injecting, shotcrete (Gnite), steel jacketing and externally bonded fiber composites which improves the deficiency present in individual column, beam and wall elements (Figure 1.5). In some special cases, passive energy dissipative devices such as metallic dampers are installed within the mounting system. These metallic dampers dissipate the seismic energy through inelastic deformation of the material. These devices are classified based on their yielding mechanism such as added damping and stiffness element (ADAS), honeycomb damper, Triangular ADAS and steel slit damper. In line with the above strength enhancement strategies for the existing structure, the proposed hybrid structure which consists of CLT as a partially infilled wall along the bay length and with the
metallic damper as connectors (Figure 1.4). In this system, the major role is played by the metallic damper along with the CLT. In the recent years, the research conducted by Yazdinezhad (2016) at UBC, has indicated that the potential use of CLT within the reinforced concrete structure resulted in mitigating the torsional effect. Hence, this system can be categorized under system level strengthening technique for the retrofitting method. Numerous studies have improved in the enhancement of strength and ductility of the structure by considering the system level retrofitting techniques. The choice of selection of retrofitting is based on the cost, availability and structural behavior of the element. The conventional retrofitting method such as concrete infill walls and steel bracing requires extensive man power, construction time for strengthening the existing structure and results in significant cost. In the case of proposed hybrid structure based retrofitting, the CLT and the metallic damper connectors can be installed within the frame for the required dimensions without much man power and with minimal design cost.

![Figure 1.5 System and Element Strengthening](https://example.com/image)

*Figure 1.5 System and Element Strengthening ([Kaplan and Salih, 2011] by permission through Creative Commons License)*
1.4 **Objectives**

The objectives of this research are as follows:

- To carry out the parametric investigation for the steel slit damper using Abaqus Finite Element Analysis Program.

- Develop the direct displacement-based design for the proposed CLT-RC hybrid system and analyze the designed structure with Non-linear Time History Analysis (NLTHA).

1.5 **Organization of thesis**

This thesis is organized into 6 chapters:

**Chapter 2** provides a thorough literature review about the direct displacement-based design procedure for the structural systems.

**Chapter 3** discuss the numerical modeling and the parametric investigation carried out for the metallic (steel slit) damper considered in this research.

**Chapter 4** discuss the modeling of the proposed hybrid system and their structural response is compared with the bare frame.

**Chapter 5** outlines the step by step direct displacement-based design procedure developed for the proposed hybrid system. Further, the proposed design methodology is verified for a 2D 6-storey RC frame using non-linear time history analysis.

**Chapter 6** presents the conclusion, limitations, and recommendations for the future research.
Chapter 2: Literature Review of Direct Displacement-Based Design

Seismic design of buildings can be carried out using Force-based and Displacement-based design procedures. In the force based design procedure, the seismic lateral forces are calculated by using the estimated period and mass of the structure, and the displacement is the final output of the design process. In the direct displacement-based design, the structure is designed for the user specified performance objective. The performance objectives are defined in terms performance target such as stress, strain, displacements and accelerations (Ghobarah 2001). Hence, this section briefly discuss the limitation associated in using the force-based design and an extensive literature review has been carried out on the direct displacement-based design.

2.1 Force-Based Design

The force-based design procedure is being followed in most of the worldwide seismic design codes. The design procedure starts with the calculating the fundamental period (\(T_a\)) of the structure using the empirical equation. Then, the seismic base shear is obtained by calculating the spectral acceleration corresponding to the period using the uniform hazard spectrum and multiplied by the seismic weight of the structure. The steps involved in this force-based design are as follows:

Step 1: Calculate the fundamental period (\(T_a\)) of the structure using the empirical expressions:

For RC frames,

\[
T_a = 0.075(h_n)^{0.75}
\]  

(2.1)

where \(h_n\) is the total height of the building (m).

Step 2: Calculate the elastic base shear (\(V_{\text{elastic}}\)) corresponding to the design spectral acceleration
for the obtained fundamental period using the uniform hazard spectrum and multiplied by the seismic weight of the building.

Step 3: The elastic base shear is reduced to the design base shear by appropriately selecting the force reduction factors (R). These force reduction factors are accounted to describe the inelastic behavior associated within the structural system. In Canadian code context, force reduction factors are described in terms of over strength (R_o) and ductility reduction (R_d) factors which is shown in Figure 2.1. This reduction factor will vary from the structural system and they are calculated using the equal displacement approximations. Hence, the designers should choose appropriate factors for the structural system considered. The inappropriate accounting of these factors will lead to the improper design and result in huge construction cost.

Figure 2.1 Overstrength and ductility reduction factors ([Alam et al. 2012] Adapted with permission from publisher)

By accounting the force reduction factors, the design base shear equation according to National Building Code of Canada (NBCC) is given as:

\[
V = \frac{S(T_a)M_v I_E W}{R_d R_o}
\]

(2.2)

where \(V\) is the base shear to be applied to the building; \(S(T_a)\) is the design spectrum at the design period \(T_a\); \(M_v\) accounts for the higher mode effects on the base shear; \(I_E\) is the importance factor
of the building; \( W \) is the seismic weight; \( R_d \) is the ductility factor; and \( R_o \) is the overstrength factor (NRC, 2010).

Priestley (2000) and other researchers developed a new approach for the seismic design to overcome the limitations associated with the force-based design procedure. The limitations are summarized as follows:

1. In the force-based design, the distribution of forces for the structural element is based on the estimation of initial stiffness of the structure. Since the stiffness is dependent on the strength of the elements, this cannot be known until the design process is complete.

2. Allocating seismic forces between elements based on the initial stiffness (even if accurately known) is illogical for many structures because it incorrectly assumes that the different elements can be forced to yield simultaneously.

3. The force-based design does not account for the energy dissipation capacity of different materials and structural systems. The improper quantification of force reduction factors will increase the design cost.

### 2.2 Direct Displacement-Based Design

Priestley (2000) developed direct displacement-based design procedure to overcome the weakness in the force-based design procedure. The aim of the direct displacement-based design is to design the structural system for the required performance objective. The performance objectives are defined in terms of performance target such as displacement or drift (Ghobarah 2001). This performance target differs with respect to earthquake hazard and design level. Ghobarah (2001) summarized the performance level corresponding the damage state and drift limits as shown in Table 2.1. In Canada, the structure is designed to perform within interstorey drift limit of 2.5\%, to represent the near collapse performance level.
Table 2.1 Drift limits for different performance level (Ghobarah 2001)

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Damage state</th>
<th>Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully operational, Immediate occupancy</td>
<td>No damage</td>
<td>&lt;0.2%</td>
</tr>
<tr>
<td>Operational, Damage control, Moderate</td>
<td>Repairable</td>
<td>&lt;0.5%</td>
</tr>
<tr>
<td>Life safety – Damage state</td>
<td>Irreparable</td>
<td>&lt;1.5%</td>
</tr>
<tr>
<td>Near collapse, Limited safety, Hazard reduced Collapse</td>
<td>Severe</td>
<td>&lt;2.5%</td>
</tr>
</tbody>
</table>

Direct Displacement-Based Design (DDBD) framework developed by Priestley (2000) is shown in Figure 2.2. This design starts by representing the multi-degree of freedom structure by an equivalent single degree freedom by using the substitute-structure approach (Shibata and Sozen, 1974) as design displacement, equivalent mass and effective height (Figure 2.2a). Figure 2.2 (b) presents the bi-linear envelope of the lateral force-displacement response of the SDOF and it points out that the initial elastic stiffness $K_i$ is followed by a post-yield stiffness $rK_i$. The unique feature of DDBD is that the structure is represented by the maximum displacement at secant stiffness $K_e$ and the equivalent viscous damping ($\xi_{eq}$). The equivalent viscous damping depends on the energy dissipation capacity of the structural system during its inelastic behaviour. As shown in Figure 2.2 (c), the equivalent viscous damping (EVD) is represented for the different structural types. The EVD value is taken from the graph based on the system ductility demand demand ($\mu$). Once the displacement at the maximum response and the equivalent viscous damping are known from the above steps, the effective period ($T_e$) is determined from the scaled displacement spectra. Before finding $T_e$, the displacement spectra should be scaled corresponding
to the equivalent viscous damping (Figure 2.2 d). The effective stiffness \((K_e)\) of the equivalent SDOF structure at maximum displacement can be evaluated using the effective period and equivalent mass. Finally, total base shear is determined using the effective stiffness and the design displacement.

![Image](image_url)

Figure 2.2 Fundamental design of Direct Displacement-Based Design (DDBD) ([Priestley and Kowalsky, 2000] Adapted with permission from publisher)

2.2.1 General Aspects of DDBD

The direct displacement-based design procedure starts by representing the multi-degree of freedom structure by an equivalent single-degree of freedom using the inelastic mode response of the structure. The fundamental equations involved in the design process are provided below:

(a) Design Displacement Profile

The design displacement profile of the frame are derived from the normalized inelastic mode
shape $\delta_i$, where $i = 1$ to $n$ are the storey, and the displacement shape $(\Delta_i)$ of the structure is given by the relationship accounting the critical storey displacement $(\Delta_c)$ as:

$$\Delta_i = \delta_i \left( \frac{\Delta_c}{\delta_c} \right)$$

(2.3)

where the normalized inelastic mode shape depends on the height $(H_i)$ and roof height $(H_n)$ according to the following relationships:

for $n \leq 4$: $\delta_i = \frac{H_i}{H_n}$

(2.4)

for $n > 4$: $\delta_i = \frac{4}{3} \left( \frac{H_i}{H_n} \right) \left(1 - \frac{H_i}{4H_n}\right)$

(b) Multi-Degree of Freedom (MDOF) to Equivalent Single Degree of Freedom Representation

Design Displacement $(\Delta_d)$

$$\Delta_d = \sum_{i=1}^{n} \left( m_i \Delta_i^2 \right) / \sum_{i=1}^{n} \left( m_i \Delta_i \right)$$

(2.5)

Equivalent mass $(m_e)$

$$m_e = \sum_{i=1}^{n} \left( m_i \Delta_i \right) / \Delta_d$$

(2.6)

Effective height $(H_e)$

$$H_e = \sum_{i=1}^{n} \left( m_i \Delta_i H_i \right) / \sum_{i=1}^{n} \left( m_i \Delta_i \right)$$

(2.7)

(c) Design ductility

The design ductility is expressed as the ratio of design displacement $(\Delta_d)$ to the yield displacement $(\Delta_y)$.

$$\mu = \frac{\Delta_d}{\Delta_y}$$

(2.8)
The yield displacement is established by considering the yield drift of the structure.

For reinforced concrete frame,

\[ \theta_y = 0.5 \varepsilon_y \frac{L_b}{h_b} \]  

(2.9)

where \( L_b \) and \( h_b \) are the beam span between column centerlines, and overall beam depth respectively, and \( \varepsilon_y \) is the yield strength of the flexural reinforcement or structural steel.

Yield displacement is given as:

\[ \Delta_y = \theta_y \cdot H_e \]  

(2.10)

(d) Equivalent Viscous Damping (\( \xi_{eq} \))

The most important part of DDBD is the equivalent viscous damping (EVD). EVD is defined as the sum of elastic \( \xi_e \) and hysteretic damping \( \xi_{hyst} \):

\[ \xi_{eq} = \xi_e + \xi_{hyst} \]  

(2.11)

Elastic Damping (\( \xi_e \))

Elastic damping is taken into consideration to account for the material nonlinearities because of early yielding and energy dissipation at initial loading cycle. Moreover, the damping accounted does not consider damping provided by the foundation and the interaction between the structural and non-structural elements. For concrete structures, elastic damping is taken as 5%.

Hysteretic Damping (\( \xi_{hyst} \))

In this design, the nonlinear response of the structure is represented by the equivalent linear structure where the maximum displacement response will be equal to the original nonlinear structure response. Initially, Jacobsen (1960) presented an approximate solution for the equivalent linear system in which steady-state response of the nonlinear oscillator is defined by the equivalent linear oscillator. The assumption made by Jacobsen (1960) is that both oscillators have the same natural frequency and dissipate equal energy per cycle of sinusoidal response.
This is called Jacobsen’s area-based approach. The hysteretic damping equation proposed by Jacobsen is given in Equation 2.12.

\[
\xi_{\text{hyst}} = \frac{1}{4\pi} \frac{E_{\text{dissipated}}}{E_{\text{stored}}} = \frac{1}{2\pi} \frac{A_{\text{hyst}}}{F_o u_o}
\]  

where \( A_{\text{hyst}} \) is the value of the area of hysteresis energy dissipated per cycle, \( F_o \) is the maximum force and \( u_o \) is the maximum displacement per cycle. Importantly, Jacobsen (1960) developed the hysteretic damping expression based on the initial stiffness of the original structure. Later, Priestley (2000) and his co-researchers adopted the equivalent linear structure approach for the direct displacement-based design in which they represented inelastic response in the terms the secant stiffness. The limitations in using Jacobsen’s equation based on the initial stiffness assumption are noted as:

1. Both systems have a same initial period \( (T_e) \), but in real cases the earthquakes will have a varied frequency content rather than the same excitation frequency.

Figure 2.3 Energy dissipated and energy stored in hysteretic cycle ([Blandon and Priestley, 2005] Adapted with permission from publisher)
2. The energy dissipation was calculated based on the one cycle criterion. This consideration ignores all cycles that take place prior to reaching the maximum displacement. Further, researchers carried out an extensive study of the equivalent viscous damping equation based on Jacobsen’s initial stiffness and with secant stiffness approximation. They found that the application of secant stiffness at the peak displacement resulted in an equal period shift in all hysteretic models. Moreover, the researcher carried out an extensive review of the various hysteretic model for the different structural systems. The general expression for the equivalent viscous damping and ductility is given by (Priestley, Calvi, and Kowalsky, 2007):

\[ \xi_{eq} = 0.05 + C \left( \frac{\mu - 1}{\mu \pi} \right) \]  

(2.13)

where \( C \) is a constant and \( \mu \) is a ductility. The “\( C \)” value is determined by conducting extensive nonlinear time history analysis for the structural systems. In some cases, equivalent viscous damping equations are derived for the different structural systems by the researchers are presented in Table 2.2.

(e) Effective Period \((T_e)\) of the substitute structure

The effective period of the structure is established by using the equivalent viscous damping and the damped displacement spectrum (Figure 2.2d). Before, finding the \( T_e \) the damped displacement spectrum should be scaled according to equivalent viscous damping obtained for the structural system. Then, by using the design displacement \((\Delta_d)\) and corresponding to scaled damped displacement spectrum, the effective period is determined.

(f) Effective stiffness \((K_e)\) of the substitute structure

Effective stiffness is calculated at the maximum displacement response of the substitute structure:
\[ K_e = 4\pi^2 m_e / T_e^2 \quad (2.14) \]

(f) **Design Base Shear** \((V_{\text{Base}})\)

Design base shear for the MDOF structure from the substitute structure is given as:

\[ F = V_{\text{Base}} = K_e \Delta_d \quad (2.15) \]

The obtained design base shear \((V_{\text{Base}})\) should be distributed to the floor levels in proportion to the product of mass and displacements as given in Equation 2.16. Then, the structure should be analyzed and designed for the lower of seismic and factored gravity load moments. Further, the designed structure can be verified by performing non-linear time history analysis.

\[ F_i = V_{\text{Base}} (m_i \Delta_i) / \sum_{i=1}^{n} (m_i \Delta_i) \quad (2.16) \]

Some of the researchers have developed direct displacement-based design procedure for different structural system such as frame-wall structure (Sullivan et al. (2006), Garcia et al. (2010)), concentric steel braced frame (K. Wijesundara and Rajeev, 2012), structural system with supplemental dampers (Pennucci et al. (2009), Sullivan (2011)) and CLT infilled with steel moment resisting frames (Bezabeh, 2014). A detailed review is conducted about the displacement based design procedure of these structural systems are discussed below.

### 2.2.2 Direct displacement-based design of frame-wall structures

Sullivan et al. (2006) initially developed a direct displacement-based design framework for RC frame wall structures and their design flow chart is shown in Figure 2.4. Further, Garcia et al. (2010) extended their research to check the applicability of design procedure for the steel frame-wall structure. The proposed displacement based design procedure for RC frame – wall structure by Sullivan et al. (2006) starts with the assignment of strength proportion between the wall and frame. The strength assignment was based on the forces expected during the formation of 1st
mode plastic mechanism. Further, the moment profile of wall and frames are calculated using the base shear proportion and subsequently, inflection height is determined. The design displacement profile for the RC-wall structure is given as the sum of elastic displacement profile of the wall together with the displacement profile associated with the yield curvature of the wall. This study mainly considers the wall yield curvature for finding the yield displacement profiles because it governs the overall response of the structure.

\[ \Delta_i = \Delta_{y,i} + \left( \theta_d - \frac{\phi_{y,wall} h_{inf}}{2} \right) h_i \]  

(2.17)

\[ \Delta_{y,i} = \frac{\phi_{y,wall} h_{inf} h_i}{2} - \frac{\phi_{y,wall} h_{inf}^2}{6} \]  

for \( h_i > h_{inf} \)  

(2.18)

\[ \Delta_{y,i} = \frac{\phi_{y,wall} h_i^2}{2} - \frac{\phi_{y,wall} h_i^3}{6 h_{inf}} \]  

for \( h_i \leq h_{inf} \)  

(2.19)

where \( \Delta_i \) is the design displacement at level \( i \), \( \Delta_{y,i} \) is the displacement at level \( i \) at a yield of the walls, \( \theta_d \) is the design storey drift according to the code drift limit, \( \phi_{y,wall} \) is the yield curvature of the walls, \( h_{inf} \) is the inflection height and \( h_i \) is the height at level \( i \). The equivalent viscous damping for SDOF system is determined separately for the frame and wall by using a trial effective period is provided in

\[ \xi_{byst,wall} = \frac{95}{1.3\pi} \left( 1 - \frac{1}{(\mu_{wall})^{0.5}} - 0.1 r \mu_{wall} \right) \left( 1 + \frac{1}{(T_{e,trial} + 0.85)^4} \right) \]  

(2.20)

\[ \xi_{byst,frame} = \frac{120}{1.3\pi} \left( 1 - \frac{1}{(\mu_{frame})^{0.5}} - 0.1 r \mu_{frame} \right) \left( 1 + \frac{1}{(T_{e,trial} + 0.85)^4} \right) \]  

(2.21)

This trial period is determined using the relationship between the number of stories and system ductility (Grant et al. 2005). This followed by scaling the displacement spectrum with respect to
damping and then final base shear was found out. The proposed procedure also validated by performing non-linear time history analysis.

![Figure 2.4 Frame-wall structure ([Sullivan et al. 2006] Adapted with permission from publisher)](image)

Garcia et al. (2010) used the displacement design procedure framework proposed by Sullivan in 2006 and checked the applicability of this procedure for the steel frame with the reinforced concrete wall as shown in Figure 2.6. The displacement profile and equivalent viscous damping are implemented by assigning the shear strength proportion to walls and frames. From these strength proportions, the moment and inflection height of walls are calculated to determine the design and yield displacement profile. Moreover, equivalent viscous damping expression is developed for this hybrid system by considering ductility demand of the wall and frame which is provided in Table 2.2. The authors validated the proposed design by performing the nonlinear time history analysis for 20-storey structures and concluded that design framework is effective in controlling the deformations.
Figure 2.5 Direct Displacement based design framework for frame-wall structures ([Sullivan et al. 2006]
Adapted with permission from publisher)
2.2.3 Direct displacement-based design of structural system with additional dampers

Pennucci et al. (2009) developed a displacement based design procedure for the post-tensioned concrete wall with the additional dampers. This post-tensioned hybrid system is called as “PRESS” Technology. The normal post-tensioned concrete wall produces negligible residual deformation and with no energy dissipation capacity. To enhance the energy dissipation capacity of the concrete wall, this system was equipped with displacement dependent dampers as shown in Figure 2.7. In this design, a variable constraint lambda (less than 1) is introduced to ensure a negligible deformation for the structure. The variable constraint lambda is defined as the ratio of prestressing bending resistance contribution to dampers bending resistance contribution based on the flag shaped hysteretic envelope. The previous study conducted by Rahman and Sritharan (2006) also noted that the displacement-based design developed for the rocking wall systems provided a low base shear and economically feasible design. In the design process, the displacement profile of the precast wall is obtained by summing the yield and plastic deformation of the wall. Further, the yield displacement shape of the wall is obtained using the iterative approach. Equivalent viscous damping is expressed in terms of ductility and design.
constraint lambda. The lambda value is obtained from Priestley et al. (2007). The proposed design was validated for five different number of storey building using Ruaumoko program. The result indicates that the design is in close agreement with the target drift ratio but in some storey levels the result was over estimated. The authors concluded that the reduction in drift is associated with the uncertainty between the chosen time histories records.

Sullivan (2011) developed the displacement design procedure for the 8-storey commercial building equipped with RC wall-steel EBF dual system with added dampers (Figure 2.8). The system is equipped with a damper to ensure there is no residual deformation. The design procedure is developed by assigning strength proportion to the walls and frames to obtain the design displacement profile for this structure. The authors developed the design displacement profile by considering the yield curvature of the wall only and displacement contribution from the EBF is neglected as it is connected from the ground to roof level. The equivalent viscous damping relation is established with the shear strength proportion assigned between the wall and frame, the equation can be referred from Table 2.2. The damper contribution to the viscous
damping is considered within the EVD equation as 3 times the shear force assignment of EBF. The developed design was validated by performing the nonlinear time history analysis for the 8-storey structure.

![Diagram of RC wall-steel EBF with added damper](image-url)

**Figure 2.8** RC wall-steel EBF with added damper ([Timothy J. Sullivan 2011] Adapted with permission from publisher)

### 2.2.4 Direct displacement-based design of steel concentric braced frames

Wijesundara and Rajeev (2012) developed a direct displacement-based design procedure for steel concentric braced frame structures. In the design process, design displacement profile is calculated by using normalized inelastic mode shape and yield displacement of the concentrically braced frame is obtained by considering both axial deformation and rigid rotation of the tension braces as shown in Figure 2.9.

\[
\Delta_{y,i} = \Delta_{y,i} + \Delta_{r,y,i} = \left( \frac{\varepsilon_y}{\sin \alpha \cos \alpha} \right) h_y + (\beta \varepsilon_{yc} h_y) \tan \alpha
\]

(2.22)

where \( \Delta_{xy,i} \) is the lateral displacement induced by the sway mechanism at yielding of the \( i^{th} \) storey tension brace, \( \alpha \) is the angle of brace to the horizontal line, \( \Delta_{ry,i} \) is the yield displacement of the tension brace due to rigid rotation, \( \varepsilon_{yc} \) is the yield strain of the column steel
material, $\beta$ is the ratio of the design axial force to the yielding force of the column section at $i^{th}$ storey and $h_i$ is the storey height.

![Figure 2.9 Axial and rigid rotation of braces ([Wijesundara and Rajeev 2012] Adapted with permission from publisher)](image)

The equivalent viscous damping for this structure is expressed as a function of the ductility and the non-dimensional slenderness ratio ($\lambda$) can be seen in Table 2.2. The displacement based design process for the steel concentric braced frame is shown in Figure 2.11. The proposed design procedure is validated by performing non-linear time history analysis for 4- and 8-storeys CBF structures. Non-linear dynamics analysis was conducted using 7 earthquake records and the results indicate that lower stories provide high shear values than the designed shear.

![Figure 2.10 Concentrically braced frame ([Wijesundara and Rajeev 2012] Adapted with permission from publisher)](image)
Malekpour et al. (2013) developed a direct displacement-based design procedure for steel-braced reinforced concrete frame system. The design process starts with the assignment of strength proportion of frames and braces. The design displacement profile of this structure was
characterized by the yield displacement of braces and the frames separately. Based on the
displacement profile, equivalent SDOF characteristics for this system are evaluated and
equivalent viscous damping was determined as a function of ductility and effective period using
Brandon’s method. Furthermore, equivalent viscous damping for SDOF equation is expressed as
damping of braces and frame and their corresponding overturning moment. The proposed design
was also validated for 4-, 8- and 12-storey models and authors found that 12-storey models
exceeded the design interstorey drift ratio of 2.5%, whereas 4- and 8-storey models satisfied the
target performance level.

2.2.5 Direct displacement-based design of regular steel moment resisting frames

In this research, the authors developed a Direct Displacement-Based Design (DDBD) procedure
for steel moment-resisting frames. The DDBD framework is based on the displacement spectrum
of the Iranian code of Practice. This paper follows the same design steps proposed by Priestley
(2003) except few steps in the calculations of design displacement and maximum displacement
profile. The design displacement is obtained using the yield drift equation proposed by Gupta
and Krawinkler (2002) and the maximum displacement profile for steel moment-resisting frame
is obtained by using the statistical analysis proposed by Kravasilis et al. (2006). Further, the
equivalent viscous damping is calculated using the Ramberg-Osgood (RO) model. The proposed
modification in the design is validated by performing non-linear time history analysis for 4-, 8-, 12-
and 16-stories. The results indicate that the design procedure behaves well with the chosen
performance indicators.

2.2.6 Direct displacement-based design of CLT-infilled steel moment resisting frame

Bezabeh (2014) developed an iterative direct displacement for the CLT-infilled steel moment
resisting frames (Figure 2.12). In this research, the author conducted an extensive non-linear time
history analysis for 243 single bay single storey hybrid model and established the equivalent viscous damping expression for the hybrid system using the Jacobsen’s area based approach. In the developed equation for the equivalent viscous damping and ductility, the constant “C” was represented as a complex interaction of modeling variables using the response surface method. The displacement based design process starts with the assumption of modeling variables for the CLT-infilled steel moment resisting frame. Then, strength proportion was assigned between the frame and CLT to establish the shear and moment profile. Further, the design process was identical to the design of steel moment resisting frame except the ductility and equivalent viscous damping relationship. The system ductility is expressed as a weighted between the overturning moment resistance and ductility of CLT and frame. After distributing the obtained base shear based on the strength proportion, the steel moment resisting frame was designed according to Canadian Standard. Further, the proposed framework was tested for the 3-, 6-, and 9-storey hybrid building by conducting a nonlinear time history analysis with twenty ground motion. The author concluded that the proposed framework was effective in controlling the drift within the specified limit.
Figure 2.12 Direct displacement-based design of CLT-infilled Steel Moment Resisting Frame
### 2.2.7 Summary of Equivalent Viscous Damping Equations

Table 2.2 Equivalent viscous damping equations for different structural systems

<table>
<thead>
<tr>
<th>Authors</th>
<th>Equivalent Viscous damping</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Priestley and Kowalsky (2000)</td>
<td>$\xi_{eq} = 0.05 + 0.444 \left( \frac{\mu - 1}{\mu \pi} \right)$</td>
<td>Concrete wall building, bridges</td>
</tr>
<tr>
<td>Priestley and Kowalsky (2000)</td>
<td>$\xi_{eq} = 0.05 + 0.565 \left( \frac{\mu - 1}{\mu \pi} \right)$</td>
<td>Concrete Frame</td>
</tr>
<tr>
<td>Priestley (2007)</td>
<td>$\xi_{eq} = 0.05 + 0.577 \left( \frac{\mu - 1}{\mu \pi} \right)$</td>
<td>Steel frame</td>
</tr>
<tr>
<td>Sullivan et al. (2006)</td>
<td>$\xi_{hyst,frame} = \frac{120}{1.3\pi} \left[ 1 - \frac{1}{(\mu_{frame}^{0.5})} - 0.1r_{mu_{frame}} \right] \left[ 1 + \frac{1}{(T_{e,trial} + 0.85)^{4}} \right]$</td>
<td>RC Wall-Concrete Frame structure</td>
</tr>
<tr>
<td></td>
<td>$\xi_{hyst,wall} = \frac{95}{1.3\pi} \left[ 1 - \frac{1}{(\mu_{wall}^{0.5})} - 0.1r_{mu_{wall}} \right] \left[ 1 + \frac{1}{(T_{e,trial} + 0.85)^{4}} \right]$</td>
<td></td>
</tr>
<tr>
<td>Garcia et al. (2010)</td>
<td>$\xi_{frame} = 24.9 \left[ 1 - \frac{1}{(\mu_{frame}^{0.527})} \right] \left[ 1 + \frac{1}{(T_{e,trial} + 0.761)^{1.250}} \right]$</td>
<td>Steel Frame-RC wall structure</td>
</tr>
<tr>
<td></td>
<td>$\xi_{wall} = 18.3 \left[ 1 - \frac{1}{(\mu_{wall}^{0.588})} \right] \left[ 1 + \frac{1}{(T_{e,trial} + 0.848)^{1.607}} \right]$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\xi_{SDOF} = \frac{M_{wall}\xi_{wall} + M_{OIT,frame}\xi_{Frame}}{M_{wall} + M_{OIT,Frame}}$</td>
<td></td>
</tr>
<tr>
<td>Wijesundara et al. (2011)</td>
<td>$\xi_{csw} = 0.03 + \left( 0.23 - \frac{\lambda}{15} \right) (\mu - 1) \quad \mu \leq 2$</td>
<td>Steel concentric braced frame structures</td>
</tr>
<tr>
<td></td>
<td>$\xi_{CBF} = 0.03 + \left( 0.23 - \frac{\lambda}{15} \right) \quad \mu \geq 2$</td>
<td></td>
</tr>
<tr>
<td>Pennucci et al. (2009)</td>
<td>$\xi_{F3,1.25} = 0.05 + 0.524 \left( \frac{\mu - 1}{\mu \pi} \right)$</td>
<td>Precast wall with additional dampers.</td>
</tr>
<tr>
<td></td>
<td>Flag shaped hysteretic rules are followed and the equation is valid only for $\lambda = 1.25$.</td>
<td></td>
</tr>
<tr>
<td>Sullivan (2011)</td>
<td>[ \xi_{sys} = \frac{2V_{wall}\xi_{wall} + 2V_{EBF}\xi_{EBF} + F_{\text{damper}}}{2V_b} ]</td>
<td>( \xi_{wall} ) should be taken as a concrete wall, ( \xi_{EBF} ) should be taken as 2% assuming it elastic damping of steel structures and ( F_{\text{damper}} ) as ( 3xV_{EBF} ).</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
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</tr>
</tbody>
</table>
| Bezabeh (2014) | \[ \mu_{sys} = \frac{M_{CLT}\mu_{CLT} + M_{\text{frame}}\mu_{\text{frame average}}}{M_{\text{frame}} + M_{CLT}} \]
\[ \xi_{eq} = 0.03 + C \left( \frac{\mu_{sys} - 1}{\mu_{sys} \pi} \right) \]
\[ C = 0.43 + 0.5 \times A + 0.015 \times B + (1.27 \times 10^{-3} C_t) + (3.75 \times 10^{-4} \times D) \]
\[-0.054 \times E - (2.74 \times 10^{-4} A \times C_t) - (1.01 \times 10^{2} A \times D) \]
\[-(0.019 \times A \times E) + (2.62 \times 10^{-3} B \times D) -(3.2 \times 10^{-2} \times C_t \times D) - 0.135 \times A^2 \]
\[-(1.045 \times 10^{-4} \times B^2) - (1.98 \times 10^{-8} \times C_t^2) + (5.9 \times 10^{-3} \times E^2) \] | EVD-Ductility relationship have been developed for the steel-CLT hybrid system \( C \) is given as nonlinear function of modeling variables: bracket spacing (A, m), gap between steel frame and CLT infill (B, mm), panel thickness (\( C_t \), mm), panel crushing strength (D, MPa) and post stiffness yield ratio of steel member (E, \%). |
Chapter 3: Numerical Investigation of steel slit damper

This chapter aims to carry out the numerical investigation of the steel slit damper. As the system is equipped with metallic damper as a connection to dissipate the seismic energy, a better understanding of the inelastic behavior of this metallic connection is needed. In this study, a conventional steel slit damper was numerically modeled using Abaqus finite element program and the performance was validated with the experimental result. Then, a parametric study was carried out by the factorial design of experiments to find the optimal yield load and stiffness of the damper. In addition to that, parametric interaction was also studied by conducting a response surface method for the chosen response parameters. The parametric study helped us to identify the appropriate configuration of the damper connection with the hybrid system.

3.1 Modeling of metallic damper

3.1.1 Metallic Damper

Metallic dampers are a passive system or passive energy dissipation devices that are implemented in the structures to protect against seismic excitation. These dampers are rate independent devices, meaning “the restoring force in the devices is a function of relative displacement and therefore, it dissipates the seismic input through inelastic deformation of the materials” (Symans et al. 2008). The metallic dampers can be categorized based on axial, flexural and shear yielding mechanisms such as added damping and stiffness (Bergman et al. 1987), triangular added damping and stiffness (Tsai et al. 1993), honeycomb damper (Kobori et al. 1992) and steel slit damper (Chan et al. 2008).

Though all dampers were incorporated within the bracing system of the structural frame, the steel slit damper can be used as a connection element between the beam and column junction in steel frames because of its high stiffness and excellent energy dissipating capacity (Oh et al.
This device dissipates the energy through flexural yielding of strips. The strips are made by cutting a number of slits along the longer direction of the damper. This study aims at finding the effect of different parameters on slit dampers using the design of experiment technique.

In recent years, several investigations have been made in finding the appropriate design of slit damper. Chan et al. (2008) developed a slit damper with a different strip configuration and performed a low cycle fatigue test. The result showed that the damper has good energy dissipation capacity but the damage is brittle in nature (Figure 3.1). Ghabraie et al. (2010) identified that the brittle failure is due to accumulation of stress concentration in the strip ends and optimized the shape of a strip. Further, Lee et al. (2015) proposed three non-uniform steel strip dampers (dumbbell-shaped strip, tapered strip and hourglass shape strip) which showed increased cyclic performance with stable hysteretic behaviour and cracks were evenly distributed along the strips. Teruna et al. (2015) developed four steel plates with a different configuration to minimize the stress concentration by rounding the end of slits. Although experimental results from the previous study indicated very stable hysteretic behaviour, there is a lack of research on the optimal shape of the damper with regards the height to width ratio (Lee et al. 2015; Teruna et al. 2015).

![Figure 3.1 Damage at the strip ends due to stress concentration (Chan and Albermani, 2008)](image)

Adapted with permission from publisher.
3.1.2 Model Description

This section will discuss the modeling of the damper using the Abaqus finite element program (Abaqus, 2014). In this study, a conventional steel slit damper developed by Lee et al. (2015) which has been tested experimentally is used as a baseline model for the validation of the Abaqus program. The dimension of the specimen (PSD-5) are noted as width = 500mm and height = 360mm and the typical configuration is shown in Figure 3.2. Lee et al. (2015) performed the displacement based cyclic loading to evaluate the cyclic performance of the damper. During the test, the bottom region of the specimen was bolted and top zone of the specimen (Figure 3.3) was subjected to increasing displacements at every step with the loading rate ranging from 0.1 to 0.5 mm/sec. The loading sequence applied is shown in Figure 3.4.

![Figure 3.2 Steel slit damper: Specimen Geometry](image)

Figure 3.2 Steel slit damper: Specimen Geometry
3.1.3 Finite Element Modeling

The numerical investigation is carried out using the Abaqus program to validate the experimental result. The damper is modeled using a 8-node linear brick element (C38R) with reduced integration and hourglass control as shown in Figure 3.5. As the metals usually exhibit the combined isotropic and kinematic behavior during the elastic-plastic stage, the combined isotropic and kinematic hardening parameters proposed by Chaboche (1989) is used in the material modeling.
The parameter values for SS-400 steel is obtained from Yoshida et al. (2004) and are summarized in Table 3.1. And to avoid the modeling convergence issue, mesh irregularities and overlapping, the whole element is sub-divided into different partitions. The fine meshing was adopted in the stress concentration regions to ensure the model’s accuracy. To avoid the convergence issue, the localized stress concentration regions were meshed finely to ensure the model accuracy. Figure 3.6 shows the conventional slit damper modeled in the Abaqus finite element program. The cyclic loading was applied at the top end of the specimen by using a reference point with constraint equation and bottom zone fixed in all direction. The displacement was incremented at every single step by using static general step function and the initial step size was maintained at 0.01 to avoid the convergence issues. Figure 3.7 depicts the hysteresis curve obtained for the steel slit damper modeled in the finite element model is in good agreement with the experimental results.

Table 3.1 Material Properties of steel slit damper (Yoshida et al., 2004)

<table>
<thead>
<tr>
<th>Material Grade</th>
<th>Yield stress, (MPa)</th>
<th>Q (MPa)</th>
<th>C (MPa)</th>
<th>γ (Gamma)</th>
<th>b</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson ratio (μ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS-400</td>
<td>250</td>
<td>75</td>
<td>5,880</td>
<td>55</td>
<td>13.5</td>
<td>207,000</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Q – maximum change in the size of the yield surface
C – initial kinematic hardening
γ - rate at which kinematic hardening moduli decrease with increasing plastic deformation moduli
b - rate at which the size of the yield surface changes with respect to plastic strain.
3.1.4 Damper Characteristics

After obtaining the hysteretic curves, the structural characteristics of the damper were evaluated with bilinear curve by connecting the maximum loading point at each step before the failure. From the bilinear curve, yield load \( P_y \) and yield displacement \( \delta_y \) points were found by intersecting a tangent line drawn from the initial behavior and line touching the envelope curve.
Since metals are rate independent of cyclic loading. Nonlinear properties of the damper performance are expressed in terms of equivalent stiffness \( K_{\text{eff}} \) and effective damping \( \beta_{\text{eff}} \) is calculated using Equation (3.1) and (3.2), respectively.

\[
K_{\text{eff}} = \frac{|P_{\text{max}}| + |P_{\text{min}}|}{|\delta_{\text{max}}| + |\delta_{\text{min}}|} \tag{3.1}
\]

\[
\beta_{\text{eff}} = \frac{2}{\pi} K_{\text{eff}} \left( |\delta_{\text{max}}| + |\delta_{\text{min}}| \right) \frac{E_{\text{loop}}}{|P_{\text{max}}| + |P_{\text{min}}|} \tag{3.2}
\]

where \( P_{\text{max}} \) and \( P_{\text{min}} \) – maximum and minimum loads, \( \delta_{\text{max}} \) and \( \delta_{\text{min}} \) – maximum and minimum displacements and \( E_{\text{loop}} \) – energy dissipated per cycle. The parameters required for stiffness and damping are obtained from the hysteresis envelope is illustrated in Figure 3.8.

![Figure 3.8 Stiffness and Damping characteristics](image)

**3.1.5 Parametric Study**

To reduce the stress concentration in the damper, the conventional geometrical shape is modified to find the optimal shape of the damper. A detailed parametric study has been carried out using a design of experiment technique. Three shape variables \( b_c, h \) and \( t \) with three levels were chosen by considering overall performance of the damper given in Figure 3.9. Based on the chosen variables and levels, \( 3^3 \) full factorial design of the experiment is constructed with the total number of 27 runs given in Table 3. Further, the strip width at the top \( (b) \) is made constant for all
specimens and the strip height is chosen based on the aspect ratio \(h/b_c\) which varies from 2.5 to 7.5.

**Table 3.2 Parameter and their levels considered for sensitivity analysis**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strip width at the mid-height (b_c), mm</td>
<td>9</td>
<td>18</td>
<td>36</td>
</tr>
<tr>
<td>Height of strip (h), mm</td>
<td>90</td>
<td>180</td>
<td>270</td>
</tr>
<tr>
<td>Thickness (t), mm</td>
<td>10</td>
<td>15</td>
<td>20</td>
</tr>
</tbody>
</table>

Based on the parameter combinations, 27 geometrical shapes were modeled with the validated material properties according to the design mentioned in Table 3.3. Figure 3.10, 3.11 and 3.12 show the FE model and their hysteretic responses for typical parameter combinations considering different strip heights with the same strip width at mid-height and thickness. The model developed for different combinations and their hysteretic responses are provided in Appendix A. After subjecting the models under cyclic loading, the damper characteristics were computed from the hysteretic responses summarized in Table 3.3. From Figure 3.10, the hysteretic response reveals that the load increased abruptly with a small displacement and the specimen fails at an early stage. The reason for this failure is due to increased stiffness of the specimen.
Figure 3.11 and Figure 3.12 show the response from the specimen with increased strip height which provides the stable hysteretic envelope by undergoing larger deformation and enhances the energy dissipation capacity. On comparing the hysteretic responses of three different strip heights, the specimen with h=270mm yielded around 65 kN and exhibited a stable hysteretic behaviour until the displacement of 34mm. The stress concentration in the conventional damper is not evenly distributed and concentrated only at the strip ends, which leads the specimen to fail at an early stage. If the damper shape is modified by reducing the width at the mid-height (Figure 3.10, 3.11 and 3.12), the stresses are distributed throughout the strip height and avoids the stress localization. Because of this effect, the cracks will propagate along the strip height rather than concentrating at the strip ends.

Figure 3.10 FE model and Hysteretic response for damper with b_c = 18mm, h = 90mm and t = 10mm
In Table 3.3, ductility ($\mu$) is expressed as the ratio of the maximum displacement to the yield displacement. From the different parameter combinations, the ductility of the specimens ranges from 5 to 68. The higher values of ductility are obtained from the specimen with a thickness of 15 and 20mm. The total energy absorption capacities ($E$) of the specimens were obtained by adding the loop area of the load-displacement hysteretic curve. On comparing energy absorption from the different geometrical shapes, the $E$ value ranges from 1.2 to 7 times that of the conventional slit damper.
### Table 3.3 Summary of the damper characteristics

<table>
<thead>
<tr>
<th>No.</th>
<th>Factors</th>
<th>$P_y$ (kN)</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$P_{\text{min}}$ (kN)</th>
<th>$\delta_y$ (mm)</th>
<th>$\delta_{\text{max}}$ (mm)</th>
<th>$\delta_{\text{min}}$ (mm)</th>
<th>$\mu$</th>
<th>$K_{\text{eff}}$ (kN/mm)</th>
<th>$E$ (kN-mm)</th>
<th>$\beta_{\text{eff}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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</tbody>
</table>
3.1.6  Response Surface Method (RSM)

To reduce the stress concentration at the end of the damper and to increase the damper performance, a parametric study has been carried out by choosing the strip width at mid height \( (b_c) \), the height of the strip \( (h) \) and thickness \( (t) \) as critical variables. A response surface technique is applied to generate the regression equation for the effective stiffness and effective damping as a function of design variables \( (b_c, h \text{ and } t) \). A second-degree polynomial of the form shown in Equation 3.3 is used to set up relationship between effective stiffness, effective damping and shape variables \( x \), (Montgomery, 1997):

\[
Y = \beta_0 + \sum_{i=1}^{n} \beta_i x_i + \sum_{i=1}^{n} \beta_{ii} x_i^2 + \sum_{i=1}^{n} \sum_{j=1}^{n} \beta_{ij} x_i x_j
\]  

(3.3)

where \( Y \) is response parameter, and \( \beta_0, \beta_i, \beta_{ii}, \beta_{ij} \) are the regression coefficients. From the Analysis of Variance (ANOVA), the effective stiffness (Equation 3.4) and effective damping (Equation 3.5) is expressed in terms of the design parameter with the regression coefficients given as:

\[
K_{\text{eff}} = 5.13 + 1.05A - 24.17B - 6.73C - 0.79AB + 0.21AC + 10.91BC - 1.59A^2 + 20.92B^2 + 3.02C^2
\]  

(3.4)

\[
\beta_{\text{eff}} = 20.82 + 0.29A - 3.26B + 2.30C + 0.20AB + 0.68AC - 5.03BC - 1.09A^2 - 7.49B^2 + 1.84C^2
\]  

(3.5)

where \( A, B \) and \( C \) are the strip width at mid-height \( (b_c) \), the height of the strip \( (h) \) and thickness of the damper \( (t) \), respectively and their second order interaction effects are given as \( (A^2, B^2, C^2, AB, AC \text{ and } BC) \). From Equation 3.4, it is inferred that the height of the strip is the most influencing factor for the effective stiffness and from Equation 3.5, both heights of strip and thickness contribute to effective damping. Also, higher order and interaction effects between the
height of strip and thickness have an important role in estimating effective stiffness and damping. Figure 3.13 (a) to (f) present the three-dimensional response surface plot for responses (effective stiffness and damping) and shape parameters (b, h and t). From Figure 3.13 (a) and (c), it is noted that, when the strip height (h) is between 180 and 90mm, effective stiffness increases gradually. In this region, the specimen fails at very low displacement with a high load irrespective of the b, h and t of the specimen. At the same time, Figure 3.13 (e) indicates that by decreasing b around 18mm, the specimen’s effective stiffness is reduced and performs well for the larger displacement. Figure 3.13 (b) shows that, the specimen exhibits high damping when the height of the strip is around 180mm with varying b. Also, from Figure 3.13 (f) it is evident that changing the thickness can contribute to an increase in the damping value for the specimen with different b. Further, based on the ANOVA, the importance level of each factor is determined with F-test using Design Expert Software. The factor’s F-Value is calculated by dividing the model mean square with the residual mean square. The F-Value and the corresponding P-value with the alpha level at 0.05 are shown in Table 3.4. If the P-value is less than 0.05, the factors are significant.

<table>
<thead>
<tr>
<th>Effect</th>
<th>$K_{\text{eff}}$ (kN/mm)</th>
<th>$\beta_{\text{eff}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F-Value</td>
<td>P-Value</td>
</tr>
<tr>
<td>A</td>
<td>0.500</td>
<td>0.4884</td>
</tr>
<tr>
<td>B</td>
<td>259.690</td>
<td>0.0001</td>
</tr>
<tr>
<td>C</td>
<td>20.130</td>
<td>0.0003</td>
</tr>
<tr>
<td>AB</td>
<td>0.200</td>
<td>0.6630</td>
</tr>
<tr>
<td>AC</td>
<td>0.0140</td>
<td>0.9069</td>
</tr>
<tr>
<td>BC</td>
<td>35.880</td>
<td>0.0001</td>
</tr>
<tr>
<td>A$^2$</td>
<td>0.290</td>
<td>0.5972</td>
</tr>
<tr>
<td>B$^2$</td>
<td>66</td>
<td>0.0001</td>
</tr>
<tr>
<td>C$^2$</td>
<td>1.380</td>
<td>0.2564</td>
</tr>
</tbody>
</table>
From Table 3.4, the height of the strip and thickness are the dominant factor in predicting the effective stiffness of the specimen, whereas, the height of the strip is the most important factor for effective damping.

Figure 3.13 (a)-(f): Response surface plot between the responses (effective stiffness and effective damping) and shape parameters ($b_c$, $h$ and $t$)
3.2 Summary

In this chapter, a detailed parametric study has been carried out by changing the shape variables of the steel slit damper. As a first step, a conventional damper was modeled in Abaqus and it was validated with the experimental result. Further, parametric study has been conducted to study the effect of shape parameters using the factorial design of experiments and interaction of these parameters was studied using the response surface method. The following results are highlighted below:

- By increasing the height of the strip enhances the damper by providing the low stiffness and stable hysteretic envelope.
- Reducing the strip width at the mid height, reduces the stress concentration at the end to avoid the brittle failure.
- From ANOVA, the P-value obtained for the effective stiffness confirmed that the strip height (h) and thickness are statistically significant factors.
- Effective damping of the specimen was in the range of 10 to 25%.
Chapter 4: Hybrid System Modeling

This section will describe the modeling of the proposed hybrid system using the portal frame. For the preliminary investigation, the experimental result reported for the reinforced concrete portal frame from the Stylianidis (2012) is considered. The RC frame model was developed using Extended Three Dimensional Analysis of Building System (ETABS) and it is matched with the experimental result by conducting the nonlinear static pushover analysis. Then, the same frame was considered with the inclusion of CLT and damper connection to understand the behavior of the proposed system. A typical damper response obtained from the Abaqus in Section 3.1.5 is also validated using ETABS. A detailed explanation of the modeling and results are presented below.

4.1 Modeling of RC Frame

To understand the modeling of the proposed hybrid system, reinforced concrete frame tested by Stylianidis (2012) is used as a baseline for the modeling in ETABS. Stylianidis (2012) tested reinforced concrete frame which is reduced to 1:3 scale from the original model under reversed cyclic loading. The overall length and height of the reduced frame are 1590mm and 960mm, respectively. The column and beam dimension, reinforcements and their material properties reported by Stylianidis (2012) for the reduced scale model are summarized in Table 4.1

<table>
<thead>
<tr>
<th>Frame element</th>
<th>Dimension (c/c) (mm)</th>
<th>Cross section (mm²)</th>
<th>Reinforcement ratio (%)</th>
<th>Concrete strength (MPa)</th>
<th>Yield stress of steel members (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>1590</td>
<td>100×200</td>
<td>1.17%</td>
<td>26.5</td>
<td>348</td>
</tr>
<tr>
<td>Column</td>
<td>960</td>
<td>150×150</td>
<td>1.01%</td>
<td>26.5</td>
<td>348</td>
</tr>
</tbody>
</table>
The model was developed in ETABS (2015) with the given columns and beam reinforcement. To assess the modeling behavior, ASCE-41 (ASCE Standard, 2006) nonlinear hinges, P-M2-M3 for column and M3 for Beam were assigned to both column and beam at both ends. A nonlinear static pushover analysis was performed using the ETABS and obtained results were matched with backbone curve of the experimental result as shown in Figure 4.1.

Further, to develop the model with the CLT infill and the damper connection, the reduced scale model was scaled to the original size as reported by Styliandis (2012). The performance of the reinforced concrete frame and with the cross-laminated timber and damper connection is studied by conducting a nonlinear static pushover analysis. The modeling of CLT and damper in ETABS are discussed below.
4.2 Modeling of CLT

In ETABS, CLT is modeled as shell element. Isotropic material property is assigned to the shell element. In the numerical model, representing the CLT material in terms of isotropic or orthotropic properties has no influence on the global behavior of the system (Hummel, 2017; Dickof, 2013). Therefore, the isotropic material properties assigned for the shell element are summarized in Table 4.2. Further, the energy dissipation is mainly contributed to the system by the connections. In this study, CLT is assumed to be fixed at the bottom using the rigid connection. The rigid connection is represented by using linear links with the high stiffness in the range of $1 \times 10^{11}$ MPa. The top portion of CLT and the beam are connected by the multilinear plastic links (springs) which represent the steel slit damper behavior.

### Table 4.2 Summary of CLT material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson ratio</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isotropic</td>
<td>9500</td>
<td>0.3</td>
<td>99</td>
</tr>
</tbody>
</table>

4.3 Modeling of damper connection

As the metallic damper exhibits the kinematic hardening behavior during loading and unloading stages. In ETABS, damper connection between CLT and the beam can be simulated using the multi-linear plastic springs with the kinematic hysteretic model. A typical kinematic hysteresis behavior from ETABS is shown in Figure 4.2. For developing this hysteresis loop, a backbone curve is defined in the multilinear plastic element section with the peak force and displacement of damper specimen obtained for every cycle and the hysteresis type should be selected as kinematic. In this study, to check the validity of the multilinear link element, the peak force and displacement backbone curve for every cycle were obtained from the hysteretic result of the
modeled damper specimen. For validation of the selected link element, the peak force and displacement of the specimen with the parameters \( b_c = 9 \text{mm}, \ h = 270\text{mm} \) and \( t = 10 \text{mm} \) from section 3.1.5 is taken and shown in Figure 4.3. During modeling the link, the local coordinates in the user interface should be defined correctly. In ETABS, the link element force displacement should be defined only in U2 local direction. If the local direction is not selected properly, it is not possible to achieve the proper response to the damper. The validation of the selected damper response from Abaqus and ETABS is shown in Figure 4.4. and it is confirmed that the multi-linear plastic spring can capture the steel slit damper behavior.

![Kinematic hysteresis model](image)

*Figure 4.2 Kinematic hysteresis model for the multilinear element from ETABS (© 2016, Computer and Structures, by permission)*
Figure 4.3 Backbone curve for multi-linear plastic link

Table 4.3 Multi-linear plastic link parameters

<table>
<thead>
<tr>
<th>Deformation (mm)</th>
<th>Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-17.86</td>
<td>-64.7</td>
</tr>
<tr>
<td>-12.85</td>
<td>-60.15</td>
</tr>
<tr>
<td>-8.63</td>
<td>-56.91</td>
</tr>
<tr>
<td>-7.11</td>
<td>-53.92</td>
</tr>
<tr>
<td>-4.67</td>
<td>-48.64</td>
</tr>
<tr>
<td>-2.32</td>
<td>-38.46</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.32</td>
<td>38.46</td>
</tr>
<tr>
<td>4.67</td>
<td>48.64</td>
</tr>
<tr>
<td>7.11</td>
<td>53.92</td>
</tr>
<tr>
<td>8.63</td>
<td>56.91</td>
</tr>
<tr>
<td>12.85</td>
<td>60.15</td>
</tr>
<tr>
<td>17.86</td>
<td>64.70</td>
</tr>
</tbody>
</table>
4.4 Nonlinear Static Pushover

The reduced scale model with the same material properties is scaled to incorporate the CLT and the connections. The frame dimension and cross section of elements for the scaled model are provided in Table 4.4.

<table>
<thead>
<tr>
<th>Frame element</th>
<th>Dimension (c/c) (mm)</th>
<th>Cross section (mm²)</th>
<th>Reinforcement ratio</th>
<th>Concrete strength (MPa)</th>
<th>Yield stress of steel members (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>4770</td>
<td>30x60</td>
<td>1.17%</td>
<td>26.5</td>
<td>348</td>
</tr>
<tr>
<td>Column</td>
<td>3080</td>
<td>45x45</td>
<td>1.01%</td>
<td>26.5</td>
<td>348</td>
</tr>
</tbody>
</table>

The bare frame was modeled using the same procedure followed in section 3.3.1. Another model was constructed with the addition of CLT and damper connections. The length of CLT was
assumed to be 50% of the bay length. The performance of the model was analyzed by conducting a nonlinear static pushover analysis with nonlinear hinges assigned at both end of the member.

![Figure 4.5 Pushover comparison between bare frame and with CLT](image)

The pushover results obtained for the bare frame and with CLT is shown in Figure 4.5. From Figure 4.5, it is observed that the frame strength with CLT and damper has increased nearly 2 times the original frame. From the frame with CLT curve, it is also noted that the initial frame stiffness has been increased and post yielding stiffness is mainly contributed by properties of the damper connection. The bilinear response from the damper increased the strength of the frame in the post stages and it is also important to note that the ductility of the system remains the same as a bare frame. Most importantly, the damper with the low yield force and low stiffness can effectively enhance the hybrid system behavior.
4.5 Summary

Based on the pushover results obtained for the bare frame and the proposed hybrid system, it is indicated that,

- The installation of CLT inside the system resulted in a significant increase in the initial lateral stiffness of the reinforced concrete frame.

- However, the effective performance of this hybrid system is contributed by the damper connections between the CLT and the beam.

- Damper connections have an important role in increasing the energy dissipation capacity of the system. In addition, the strength of the system can be increased by choosing the damper with the low yield load and stiffness.

- Further, it is noted that the strength enhancement of the system in the post behavior is associated with the damper strain hardening behaviour.
Chapter 5: Direct Displacement-Based Design (DDBD) of CLT-RC hybrid system

The motivation to use DDBD for this hybrid system has been already discussed by providing a detailed literature review in Chapter 2. This section will present the DDBD framework developed for the proposed hybrid system and the steps to be followed. Then, the proposed framework was applied to design the 6-storey RC 2D planar frame and subsequently design was validated by performing Non-Linear Time History Analysis (NLTHA) for the selected ground motion records.

5.1 Proposed DDBD framework for the CLT – RC hybrid system

The proposed DDBD framework for the CLT-RC hybrid system is studied for a 6-storey reinforced concrete frame. The overall building plan and elevation of the 2D frame of a six-storey building is shown in Figure 5.1 and Figure 5.2, respectively. The building has an outer bay length of 9m and inner bay length of 6m in an east-west direction. Since the structure is symmetry in the plan, the interior frame will be modeled as a two-dimensional structure. The building is assumed to be located on a very dense soil and soft rock (site class C) in Vancouver, Canada. The displacement-based design process developed for this hybrid system is shown in Figure 5.3.
Figure 5.1 Plan view of case study model

Figure 5.2 Elevation
Step 1: Develop Design Displacement Profile

The design displacement profile for the proposed hybrid system is established using the normalized elastic mode shape of the reinforced concrete frame. In this study, the structure is aimed to design for the interstorey drift limit of 2.5% which is recommended by National Building Code of Canada (NBCC). The 2.5% drift limit is to represent the near collapse level performance objective. The displacement profile is established using the Equation 5.1 (Sullivan, Priestley, and Calvi, 2010).

\[ \Delta_i = \omega_i \theta_i h_i \left( \frac{4H_n - h_i}{4H_n - h_i} \right) \]  

(5.1)
where $\omega_{q} = 1.15 - 0.0034H_n < 1.0$ is a reduction factor for higher mode amplification of drift, $H_n$ is the total building height, $h_i$ and $h_1$ are the heights of $i^{th}$ and $1^{st}$ level, respectively and $\theta_d$ is the code drift limit for the limit state considered.

**Step 2: Characteristics of equivalent Single Degree Of Freedom (SDOF) system**

Design displacement, effective mass, and effective height are the characteristics of equivalent SDOF system are calculated using the Equation 5.2, 5.3 and 5.4, respectively. Table 5.1 summarizes the equivalent SDOF characteristics of case study structure.

$$\Delta_d = \sum_{i=1}^{n} (m_i \Delta_i^2) / \sum_{i=1}^{n} (m_i \Delta_i) \quad (5.2)$$

$$m_e = \sum_{i=1}^{n} (m_i \Delta_i) / \Delta_d \quad (5.3)$$

$$H_e = \sum_{i=1}^{n} (m_i \Delta_i) \sum_{i=1}^{n} (m_i \Delta_i) \quad (5.4)$$

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>$h$ (m)</th>
<th>$\Delta_i$ (m)</th>
<th>$m_i$ (tonnes)</th>
<th>$m_i \Delta_i$</th>
<th>$m_i \Delta_i^2$</th>
<th>$m_i \Delta_i h_i$</th>
<th>$\Delta_d$ (m)</th>
<th>$m_e$ (tonnes)</th>
<th>$H_e$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>21.90</td>
<td>0.43</td>
<td>67</td>
<td>28.7</td>
<td>12.3</td>
<td>628.7</td>
<td>0.32</td>
<td>366.38</td>
<td>15.22</td>
</tr>
<tr>
<td>5</td>
<td>18.25</td>
<td>0.38</td>
<td>72</td>
<td>27.1</td>
<td>10.2</td>
<td>495.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>14.60</td>
<td>0.34</td>
<td>72</td>
<td>22.9</td>
<td>7.3</td>
<td>333.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>10.95</td>
<td>0.25</td>
<td>74</td>
<td>18.5</td>
<td>4.6</td>
<td>202.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>7.30</td>
<td>0.17</td>
<td>74</td>
<td>12.9</td>
<td>2.3</td>
<td>94.3</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>1</td>
<td>3.65</td>
<td>0.09</td>
<td>74</td>
<td>6.8</td>
<td>0.6</td>
<td>24.6</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Step 3: Ductility of the system**

Ductility of the system is the ratio of the design displacement ($\Delta_d$) to yield displacement ($\Delta_y$). Yield displacement is calculated by using the yield drift of the RC frame (Equation 5.5). As the
RC frame is considered with unequal bay length, the yield drift should be calculated separately with respect to the bay length (Equation 5.6 and 5.7).

\[
\Delta_y = \frac{2M_1\theta_{y_1} + M_2\theta_{y_2}}{2M_1 + M_2} \tag{5.5}
\]

where \( \theta_{y_1} \) and \( \theta_{y_2} \) are the yield drift of the beam in outer and inner bay, respectively, \( M_1 \) and \( M_2 \) are the moment in the outer and inner bay, respectively.

\[
\theta_{y_1} = 0.5\varepsilon_y \frac{L_{b_1}}{h_{b_1}} \tag{5.6}
\]

\[
\theta_{y_2} = 0.5\varepsilon_y \frac{L_{b_2}}{h_{b_2}} \tag{5.7}
\]

where \( L_{b_1} \) and \( L_{b_2} \) are the Length of outer and inner bay, respectively, \( \varepsilon_y \) is the yield strain of steel (0.002), \( h_{b_1} \) and \( h_{b_2} \) are the depth of beam in outer and inner bay, respectively. For the case study, the beam depth in all bays is kept constant at 500mm and moment capacities of outer and inner frames are made equal \( M_1 = M_2 \). The total overturning moment contribution from outer and inner bays does not have great influence, the relative strength of beams in outer and inner bays can be assumed any ratio (Priestley et al., 2007). For \( h_{b_1} = h_{b_2} = 500\text{mm} \) and \( M_1 = M_2 \), from Equation 5.6 and 5.7, yield drift of outer and inner bay computed as 0.0198 and 0.0132, respectively. The yield displacement of each floor is assumed to be same and it is calculated as 0.26. The ductility of the entire structure is calculated as 1.19.

**Step 4: Equivalent Viscous Damping**

As the energy dissipation capacity of this proposed system is enhanced with the use of metallic damper connection. The equivalent viscous damping of the reinforced concrete frame combined with metallic damper produces the degrading bilinear hysteretic envelope. The hysteretic
damping of the system with metallic damper (Equation 5.8) is derived based on the Jacobsen’s equation is given as (Nielsen and Imbeault, 1970).

\[
\xi_h = \frac{2(1 - \beta)\{\mu - \mu^\alpha (1 - \beta + \mu\beta)\}}{\pi\mu(1 - \beta + \mu\beta)(1 - \beta\mu^\alpha)}
\]  

(5.8)

where \( \beta \) is the post-yielding stiffness of the initial elastic stiffness, \( \alpha \) is the unloading stiffness degradation parameter (0 < \( \alpha \) < 1) and \( \mu \) is the system ductility. For reinforced concrete, \( \alpha \) and \( \beta \) can be taken as 0.4 and 0.1, respectively. The total equivalent viscous damping of the system is determined by summing up the elastic damping for the reinforced concrete as 5% and hysteretic damping. The equivalent viscous damping of the system is 11.38%.

**Step 5: Effective Period of the system**

The effective period of the system is calculated by using the damped displacement spectrum as shown in Figure. Initially, the damped displacement spectrum is scaled according to the equivalent viscous damping obtained in Step 4 using the scaling factor \( (\eta) \) as given in Equation 5.9 (Sullivan et al. 2006). With respect to the design displacement obtained in step 1, the corresponding effective period is 2.84 sec (Figure 5.4).

\[
\eta = \frac{10}{\sqrt{5 + \xi_{eq}}}
\]  

(5.9)
Step 6: Effective Stiffness and design base shear

Effective Stiffness and design base shear of the system is calculated using the Equation 5.10 and 5.11 as 1791 kN/m and 571 kN, respectively.

\[ K_e = \frac{4\pi^2 m_e}{T_e^2} \]  \hspace{1cm} (5.10)

\[ F = V_{Base} = K_e \Delta_d \]  \hspace{1cm} (5.11)

Step 7: Base shear distribution based on strength proportion and structural analysis

For this case study, the above design base shear is obtained by considering without the CLT infill contribution. If we introduce the CLT and the damper within RC frame, the design base shear will be shared between the frame and CLT. Therefore, the frame should be analyzed and designed based on the strength proportion between frame and CLT. If the strength proportion is not assigned between the frame and CLT, it will reduce the overall system response drift level.
Considering the above reasons and research performed with the frame wall type structures [Sullivan et al. (2006); Garcia et al. (2010)], the strength proportion between the frame and the CLT is taken as 40% and 60%, respectively. The base shear obtained in Step 6 is distributed in proportion between the frame \(V_{\text{frame}}\) and CLT \(V_{\text{CLT}}\) is summarized in Table 5.2.

### Table 5.2 Strength Proportion between Frame and CLT

<table>
<thead>
<tr>
<th>Storey</th>
<th>(h) (m)</th>
<th>(\Delta_i) (m)</th>
<th>(m_i) (tonnes)</th>
<th>(m_i\Delta_i)</th>
<th>(F_i) (kN)</th>
<th>(V_{\text{frame}}) (kN)</th>
<th>(V_{\text{CLT}}) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>21.9</td>
<td>0.43</td>
<td>67</td>
<td>28.7</td>
<td>140.4</td>
<td>56.1</td>
<td>84.2</td>
</tr>
<tr>
<td>5</td>
<td>18.25</td>
<td>0.38</td>
<td>72</td>
<td>27.1</td>
<td>132.7</td>
<td>53.1</td>
<td>79.6</td>
</tr>
<tr>
<td>4</td>
<td>14.6</td>
<td>0.32</td>
<td>72</td>
<td>22.9</td>
<td>111.7</td>
<td>44.7</td>
<td>67.0</td>
</tr>
<tr>
<td>3</td>
<td>10.95</td>
<td>0.25</td>
<td>74</td>
<td>18.5</td>
<td>90.4</td>
<td>36.2</td>
<td>54.3</td>
</tr>
<tr>
<td>2</td>
<td>7.3</td>
<td>0.17</td>
<td>74</td>
<td>12.9</td>
<td>63.2</td>
<td>25.3</td>
<td>37.9</td>
</tr>
<tr>
<td>1</td>
<td>3.65</td>
<td>0.09</td>
<td>74</td>
<td>6.8</td>
<td>33.0</td>
<td>13.2</td>
<td>19.8</td>
</tr>
</tbody>
</table>

The structural analysis is performed by using ETABS and subsequently, the design sections were obtained. Importantly, Priestley (Priestley et al. 2007) in his book stated that the structure should be designed based on the higher of seismic and gravity moments. When the design is carried out with the inclusion of gravity and seismic moments, will increase the section strength and reduces the response drift levels below the target values. Hence, the structural design is carried out by considering the seismic action only using CSA A23.3 (CSA. A23.3-14, 2014). The capacity design procedure should be carried out separately by accounting the moment actions from the wall. Therefore, the capacity design for this proposed system is not studied and it is outside the scope of this thesis. The designed cross-section summary for the frame with CLT and the bare frame is shown in Table 5.3 and Table 5.4, respectively.
### Table 5.3 Column and Beam Summary for Frame with CLT infill

<table>
<thead>
<tr>
<th>Storey</th>
<th>Column Description</th>
<th>Size</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,2</td>
<td>C1</td>
<td>350×350</td>
<td>8-25M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>350×350</td>
<td>6-25M</td>
</tr>
<tr>
<td>3</td>
<td>C1</td>
<td>350×350</td>
<td>6-25M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>350×350</td>
<td>6-20M</td>
</tr>
<tr>
<td>4</td>
<td>C1</td>
<td>300×300</td>
<td>6-25M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>300×300</td>
<td>6-20M</td>
</tr>
<tr>
<td>5,6</td>
<td>C1</td>
<td>300×300</td>
<td>4-25M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>300×300</td>
<td>4-20M</td>
</tr>
</tbody>
</table>

### Table 5.4 Column and Beam Summary for bare frame

<table>
<thead>
<tr>
<th>Storey</th>
<th>Column Description</th>
<th>Size</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,2</td>
<td>C1</td>
<td>450×450</td>
<td>8-35M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>450×450</td>
<td>8-30M</td>
</tr>
<tr>
<td>3</td>
<td>C1</td>
<td>450×450</td>
<td>8-30M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>450×450</td>
<td>6-25M</td>
</tr>
<tr>
<td>4</td>
<td>C1</td>
<td>400×400</td>
<td>8-30M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>400×400</td>
<td>6-25M</td>
</tr>
<tr>
<td>5,6</td>
<td>C1</td>
<td>400×400</td>
<td>6-25M</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>400×400</td>
<td>4-25M</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 to 3</td>
<td>B1</td>
<td>B2, B3</td>
</tr>
<tr>
<td></td>
<td>500×400</td>
<td>2-20M</td>
</tr>
<tr>
<td>4</td>
<td>B1</td>
<td>B2, B3</td>
</tr>
<tr>
<td></td>
<td>500×400</td>
<td>2-20M</td>
</tr>
<tr>
<td>5,6</td>
<td>B1, B2, B3</td>
<td>500×400</td>
</tr>
</tbody>
</table>
Step 8: Calculate the storey stiffness of the structure

To incorporate the CLT and damper within the structure, storey stiffness is calculated for the frame based on the force vectors using Equation 5.12. Storey stiffness of the structure is calculated as the ratio of lateral force to the relative floor displacement (Basu, D. and Reddy, P. R.M., 2016).

\[ K_f = \left( \sum_{j=i}^{n} F_j \right) / (U_i - U_{i-1}) \] (5.12)

where \( F_j \), \( U_i \) and \( K_f \) are the lateral force, floor displacement and storey stiffness of the bare frame, respectively.

Step 9: Equivalent Stiffness \((K_h)\)

The stiffness of the structure, CLT and damper are proportioned using the ratio “SR”. SR is defined as equivalent stiffness \((K_h)\) to the structural storey stiffness \((K_f)\). From the past research
conducted by Xia and Hanson (1992) indicated that the value of $SR$ should be less than 2. The value more than 2 are insignificant. Hence, SR is assumed as 1.5 for this case study.

\[
SR = \frac{K_h}{K_f}
\]  

(5.13)

Equivalent stiffness ($K_h$) is expressed as the springs in series connection for the CLT ($K_{CLT}$) and damper ($K_d$),

\[
K_h = \frac{K_{CLT}K_d}{K_f + K_d}
\]  

(5.14)

The stiffness of the CLT wall is approximated by using the Equation 5.15 (DG/TJ08-2059-2009, 2009).

\[
K_{CLT} = \frac{1}{\frac{2h^3}{3EA} + \frac{h}{1000G} + \frac{hd}{Lf}}
\]  

(5.15)

where $E =$ Modulus of elasticity of the CLT taken as 6500 (MPa), $G$ is the shear modulus (MPa), $H =$ height of the CLT in each floor (m), $L$ is the length of the wall (m), $d$ is the thickness of the CLT (m) and $f$ is the bearing capacity factor can be obtained from Canadian Wood Design Manual CSA-O86 (CSA-O86, 2014). Since equivalent stiffness can be determined from the frame storey stiffness by assuming the SR value. Therefore, the stiffness of CLT material is approximated in terms of length using the Equation 5.15. It is important to note that the stiffness may increase depending on the vertical load and the connections provided at the bottom of the wall. The future research can aim to find the influence of wall stiffness with the various connections. From the calculated wall stiffness in terms of length, the required damper stiffness is evaluated using the Equation 5.14. By establishing the damper stiffness for each floor, the user can select the number of dampers/connectors based on the dynamic characteristics and
availability of the material. Table 5.5 summarizes the about the stiffness calculations of the hybrid system.

Table 5.5 Summary of Stiffness Proportion CLT – RC hybrid system

<table>
<thead>
<tr>
<th>Storey No.</th>
<th>Force (kN)</th>
<th>Displacement (mm)</th>
<th>Storey Stiffness (Kf) (kN/m)</th>
<th>Equivalent stiffness (Kh) (kN/m)</th>
<th>Required damper stiffness (kN/m)</th>
<th>No. of Dampers reqd.</th>
<th>Damper Stiffness provided (kN/m)</th>
<th>Stiffness of CLT (kN/m)</th>
<th>CLT wall length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>56.1</td>
<td>142</td>
<td>4836</td>
<td>7254</td>
<td>15056</td>
<td>2</td>
<td>7528</td>
<td>14000</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>53.1</td>
<td>130</td>
<td>5201</td>
<td>7802</td>
<td>17622</td>
<td>2</td>
<td>8811</td>
<td>14000</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>44.7</td>
<td>109</td>
<td>5131</td>
<td>7696</td>
<td>17092</td>
<td>2</td>
<td>8546</td>
<td>14000</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>36.2</td>
<td>79</td>
<td>6789</td>
<td>10184</td>
<td>16005</td>
<td>2</td>
<td>8002</td>
<td>28000</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>25.3</td>
<td>51</td>
<td>6947</td>
<td>10421</td>
<td>16598</td>
<td>2</td>
<td>8299</td>
<td>28000</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>13.2</td>
<td>20</td>
<td>11429</td>
<td>17143</td>
<td>28965</td>
<td>3</td>
<td>9655</td>
<td>42000</td>
<td>3</td>
</tr>
</tbody>
</table>

At this stage, the displacement-based design is complete, and the frame is incorporated with the calculated CLT wall length and the damper connections in ETABS software. The modeling of the frame with CLT and damper connection was already discussed in Chapter 3. Finally, the designed 2D frame was validated by conducting Non-Linear Time History Analysis (NLTHA) with the selected ground motions in ETABS software.

**Step10: Validate through NLTHA**

To validate the proposed CLT-RC hybrid system, nonlinear time history analysis is performed with the fifty ground motions. The ground record selection was carried out using the conditional mean spectrum method (Atkinson and Goda 2011; Baker 2011). The conditional mean spectrum method accounts the multiple target spectra (with an inter-period correlation of spectral ordinates), representing distinct response spectral features of different earthquake types (i.e. crustal, interface and deep in the slab) and their relative contribution to the overall seismic hazard (Figure 5.6) (Tesfamariam et al., 2015; Atkinson and Goda, 2011). Researchers (Tesfamariam and Goda, 2015; Tesfamariam and Goda, 2017) have successfully used the
condition mean spectrum based ground motion selection in their studies. In this case study, the ground motions were selected by using the first three modes of the building. In the CMS method, anchor period ($T_1$), minimum ($T_{\text{min}}$) and maximum ($T_{\text{max}}$) period was set for the selection of ground motion. Anchor period was selected based on the fundamental period of the building. Eigen value analysis showed that the first fundamental period of the hybrid building as 3.02 seconds, hence, the anchor period was chosen as 3 seconds for ground motion selection. Then, $T_{\text{min}}$ and $T_{\text{max}}$ cut off was set as 0.5 and 4 seconds, respectively to represent the entire vibration period. The designed building is assumed to be in Vancouver of soil site class “C”. The records were scaled to the 2 % in 50 years uniform hazard spectrum of Vancouver and the seismic hazard spectral matching is shown in Figure 5.7.

![Figure 5.6 Seismic deaggregation plot for the selected ground motion (in terms of magnitude, distance and earthquake type) for 2 % PE in 50 years](image)
The 2D 6-storey frame modeled with appropriate CLT wall length and damper connections based on the stiffness proportion using ETABS is shown in Figure 5.8. The seismic performance of this design methodology is verified by the non-linear time history analysis. For the structure designed as a bare frame and with CLT, the validity of the model is checked with the set of fifty ground motions. The ground motion selected has good variability with respect to magnitude, distance and earthquake type. The seismic behaviour of the structure is monitored by computing the interstorey drift ratio for each run of the ground motions. The interstorey drift result obtained for the fifty ground motions for the bare frame and with CLT is shown in Figure 5.9 and 5.10, respectively.
Figure 5.8 2D RC-CLT Hybrid System
Figure 5.9 Interstorey drift for the bare frame

Figure 5.10 Interstorey drift for the CLT-RC hybrid system
As the structure is designed for the interstorey drift limit of 2.5%. The structure is expected to perform close to 2.5% target limit. To understand the response of the designed structure, interstorey drift is monitored for all the ground motions. The results obtained for the bare frame and with CLT is shown in Figure 5.9 and Figure 5.10, respectively. Interstorey drift results indicate that the designed bare frame and frame with CLT are performing close to interstorey drift of 2.5%. Figure 5.11 shows the average results obtained for the designed structures. These average plots are indicative of the system performance because of the ground motion selected has different earthquake levels and magnitude. From the study carried out, it is also important to note that the CLT-RC system equipped with dampers can perform well if the damper has very low yield force and stiffness. The future research can aim to reduce the damper yield force and stiffness to enhance the performance in terms of drift limit.
Chapter 6: Conclusion and Future Recommendations

This thesis developed a new CLT-RC hybrid system for the seismic retrofitting of reinforced concrete buildings. The hybrid system consists of reinforced concrete frame infilled with CLT and metallic damper connections. The advantage of using damper connections is to enhance the energy dissipation capacity of this system. First, a parametric study has been conducted using a finite element program to understand the dynamic characteristics of the damper specimens. Further, the overall seismic performance of the hybrid system was analyzed by developing a direct displacement-based design framework.

In Chapter 3 of this thesis, a conventional steel slit damper has been modeled using Abaqus and was validated with the experimental result reported by Lee et al. (2015). Then, a detailed parametric study has been carried out on the shape variables using the factorial design of experiment. During this study, 27 models were generated in Abaqus and it was subjected to cyclic loading. Further, dynamic characteristics of the specimens were evaluated from the peak force and displacement of every cycle in the hysteretic envelope. Then, the effect of shape variables and their interactions on the response parameters (effective stiffness and damping) were studied using the response surface method and ANOVA. The following findings from Chapter 3 are given below:

- Finite element model created for the conventional specimen was able to capture the kinematic hardening behavior.
- Increasing the strip height of the damper can effectively enhance the energy dissipation capacity by providing the stable hysteretic behavior.
- Decreasing the strip width at mid height reduces the stress concentration at the end.
- From the ANOVA results it is also clear that the P-value of the strip height and thickness is less than 0.05 for the responses effective stiffness and damping. Therefore, the parameters are statistically significant.

In Chapter 4 of this thesis, a preliminary investigation was conducted on the hybrid system by performing the nonlinear static pushover analysis. First, a typical connection from chapter 3 were modeled using the link element in ETABS and the results were verified. Then, the reinforced concrete frame was modeled with the appropriate damper connections and the CLT. Subsequently, a nonlinear static pushover analysis was performed and compared with the bare frame. The result indicated that the initial stiffness contribution of the system was increased by the CLT before the activation of the damper connection. Once the connections activated, the strength enhancement was observed in the post stages of the system. The stiffness associated with the post behavior was mainly contributed by the kinematic hardening behaviour of the connectors.

Finally, in chapter 5, the seismic performance of the reinforced concrete frame infilled with CLT and damper connection was studied by developing a displacement based design framework. The developed displacement design follows the same procedure as the reinforced concrete frame. Then, a strength proportion between the frame and CLT was assigned for the RC frame analysis and the design sections were obtained. Further, the required stiffness of the CLT and damper connections were determined based on the storey stiffness. Finally, the developed design methodology was used to model a 6-storey 2D frame using ETABS with the appropriate damper stiffness and CLT wall length. The designed structure was verified by conducting a non-linear time history analysis. From this study, the following findings are summarized below:
The developed displacement-based design framework successfully tested for a 6-storey 2D RC-CLT hybrid system and is successfully verified with the nonlinear time history analysis.

- The interstorey drift ratio of the designed structure reveals that the hybrid system performance lies within the 2.5% of target interstorey drift limit.
- It can be argued that the interstorey drift is not close to the 2.5% target drift. This is due to the assumption made in the stiffness proportion between the frame, CLT and the dampers.
- During the preliminary stages of this research, it is noted that by changing the stiffness assumption, the stiffness of CLT and damper connections can be reduced to make the structure to perform close to 2.5% drift.

6.1 Contributions

In recent decades, the timber-based hybrid structural system is becoming popular in Canada. In line with that, this thesis attempted to use cross laminated timber as an infill and metallic damper with reinforced concrete structure. The energy dissipation capacity for the structure is enhanced with metallic damper connectors. Based on this research, the proposed system can be used as a potential retrofitting method for the non-ductile reinforced concrete buildings. The use of CLT panels in this system has several advantages over the conventional concrete shear wall. It provides the lighter structure (i.e., attracts less seismic force), reduce the construction time, and is easy to install and cost effective. In addition to that, in the seismic design process of this hybrid system, a stiffness based proportion was used for the selecting the appropriate number of damper connector. This will enable the user to select the appropriate configuration of the damper connector based on the stiffness and number of connectors required.
6.2 Limitations

- The damper results obtained from the different parametric combination is not verified with the experimental result. The performance of the damper is evaluated with the proposed configuration from the literature and subsequently, a parametric study was conducted to obtain the dynamic characteristics results for different damper configuration.

- The stiffness equation used for determining the in-plane stiffness of the CLT wall is a representative for the design consideration and the equation does not account the effect of vertical load and the connection types. Further, future research should aim to find the in-plane stiffness of the wall by considering the effect of wall aspect ratio and connections. This will help in refining the design flow process.

- This study has tested the hybrid concept with the 6-storey structure to understand its seismic behaviour.

6.3 Future Research

- Damper connection with the CLT should be tested experimentally to find out the failure mechanism under cyclic loading.

- Low yield strength dampers can be selected as the connection type with the CLT and this may enhance the system behaviour of the proposed system.

- A parametric study can be performed by reducing the overall width and height of the damper. This might help in reducing the damper yield force and stiffness.

- A complete finite element model can be developed for this overall hybrid system. This will indicate the stress level at the connection regions and aid in identifying the stiffness associated with the CLT wall.
A finite element study can be carried out only for the CLT wall with the different connections and vertical loading combinations. This will help in developing a stiffness equation incorporating the effect of vertical load and then it can be incorporated with the proposed system design.
References


SOM. (2013). *Timber tower research project: Final report*, WoodWorks Education Lab,


Appendices

Appendix A

Figure A.1 FE model and Hysteretic response for damper with $b_c = 9$mm, $h = 90$mm and $t= 10$mm

Figure A.2 FE model and Hysteretic response for damper with $b_c = 18$mm, $h = 90$mm and $t= 10$mm
Figure A.3 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 90\text{mm}$ and $t=10\text{mm}$

Figure A.4 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 180\text{mm}$ and $t=10\text{mm}$
Figure A.5 FE model and Hysteretic response for damper with $bc = 18\text{mm}$, $h = 180\text{mm}$ and $t = 10\text{mm}$

Figure A.6 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 180\text{mm}$ and $t = 10\text{mm}$
Figure A.7 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 270\text{mm}$ and $t= 10\text{mm}$

Figure A.8 FE model and Hysteretic response for damper with $bc = 18\text{mm}$, $h = 270\text{mm}$ and $t= 10\text{mm}$
Figure A.9 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 270\text{mm}$ and $t = 10\text{mm}$

Figure A.10 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 90\text{mm}$ and $t = 15\text{mm}$
Figure A.11 FE model and Hysteretic response for damper with bc = 18mm, h = 90mm and t = 15mm

Figure A.12 FE model and Hysteretic response for damper with bc = 36mm, h = 90mm and t = 15mm
Figure A.13 FE model and Hysteretic response for damper with bc = 9mm, h = 180mm and t= 15mm

Figure A.14 FE model and Hysteretic response for damper with bc = 18mm, h = 180mm and t= 15mm
Figure A.15 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 180\text{mm}$ and $t = 15\text{mm}$.

Figure A.16 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 270\text{mm}$ and $t = 15\text{mm}$.
Figure A.17 FE model and Hysteretic response for damper with $bc = 18\text{mm}$, $h = 270\text{mm}$ and $t = 15\text{mm}$

Figure A.18 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 270\text{mm}$ and $t = 15\text{mm}$
Figure A.19 FE model and Hysteretic response for damper with $b_c = 9\text{mm}$, $h = 90\text{mm}$ and $t= 20\text{mm}$

Figure A.20 FE model and Hysteretic response for damper with $b_c = 18\text{mm}$, $h = 90\text{mm}$ and $t= 20\text{mm}$
Figure A.21 FE model and Hysteretic response for damper with $bc = 36\text{mm}$, $h = 90\text{mm}$ and $t= 20\text{mm}$

Figure A.22 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 180\text{mm}$ and $t= 20\text{mm}$
Figure A.23 FE model and Hysteretic response for damper with bc = 18mm, h = 180mm and t= 20 mm

Figure A.24 FE model and Hysteretic response for damper with bc = 36mm, h = 180mm and t= 20 mm
Figure A.25 FE model and Hysteretic response for damper with $bc = 9\text{mm}$, $h = 270\text{mm}$ and $t= 20\text{ mm}$

Figure A.26 FE model and Hysteretic response for damper with $bc = 18\text{mm}$, $h = 270\text{mm}$ and $t= 20\text{ mm}$
Figure A.27 FE model and Hysteretic response for damper with bc = 36mm, h = 270mm and t= 20 mm