

Analytical Seismic Performance Assessment of Braced Timber Frames with Shape Memory Alloy Fasteners

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

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ABSTRACT

The increasing adoption of mass timber construction requires a thorough understanding of its structural performance. Brace timber frames (BTFs) commonly serve as lateral load-resisting systems in mass timber buildings; however, further insight into their seismic performance, particularly regarding ductility and energy dissipation, remains essential. Shape memory alloy (SMA) dowels at BTF connections provide potential advantages due to their superelasticity, enabling large displacements with minimal permanent deformation. This analytical research investigates the seismic performance of BTFs with SMA dowel-type connections compared to traditional steel connections.

The methodology involves developing a numerical framework in OpenSees to capture the nonlinear behaviour of BTFs with steel and SMA dowel-type connections. The framework involves calibrating uniaxial material models at the connection level based on experimental hysteresis results. Brace-level models incorporate asymmetrical deformation at each brace end, documented in the literature. Frame-level models facilitate pushover and time history analyses. This approach bridges the gap between experimental observations in connections and structural system applications, specifically evaluating how connection-level self-centering translates into system-level performance in moderate seismic zones in Canada.

Using this numerical framework, the self-centering capacity of SMA dowel-type connections is assessed at the system-level. The seismic response of SMA-connected frames is compared to traditional steel-connected frames using BTF building prototypes. Key findings show that SMA connections exhibit superior self-centering, substantially reducing residual drift compared to steel connections, thus enhancing seismic resilience. Although SMA-connected frames experienced higher peak drifts, their minimal permanent deformations significantly reduced post-earthquake repair needs.

Numerical results indicated that SMA connections increased peak interstorey drift by 8-32% but significantly reduced residual drift by approximately 90% in both prototypes. Peak floor accelerations decreased by 3-13% with SMA connections. These findings confirm that SMA dowels effectively improve post-earthquake performance without compromising structural strength. This will guide future code developments and research directions for BTF systems incorporating SMA dowel-type connections.

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CHAPTER 1. INTRODUCTION

1.1 Background

The high strength-to-weight ratio and ease of construction, both on- and off-site, have established wood as a preferred material in modern building practices. At the same time, a global emphasis on sustainable design and carbon emissions mitigation has contributed to the increased use of wood as it is considered a renewable structural material. (IPCC, 2018). Unlike typical building materials such as cement and steel, which are associated with significant releases of greenhouse gases during their production, wood is considered to capture and store atmospheric carbon, making it an embodied carbon material (Buchanan & Levine, 1999; Gustavsson & Sathre, 2006). This renewable resource ensures a continuous supply under sustainable forestry management programs and policies. In The State of Canada's Annual Report from Natural Resources Canada (2023c), the area of forest harvested is less than 0.5% each year of the total 367 million hectares of forest land.

The growing interest in wood as a construction material has resulted in significant advances in engineered wood products (EWPs). Mass timber, a popular category within EWPs, provides designers with large sections that offer strength, dimensional stability, and fire resistance. These attributes make them appropriate structural components for building and bridge applications, which have traditionally relied on concrete and steel. Innovative wood products, alongside the efforts to produce more sustainable structures, and recent code revisions have brought mass timber to the front of the construction sector.

Recognizing these benefits, the Canadian government has supported projects intended to develop the mass timber industry. In these efforts, the Forest Innovation Program (Natural Resources Canada, 2023a) and the Green Construction through Wood Program, GCWood, (Natural Resources Canada, 2023b) are two examples of important government initiatives. This growing interest in mass timber construction, combined with advances in manufacturing technology and thorough research into product and system performance, has led to significant changes to key building codes. Notably, the most recent editions of the National Building Code of Canada (NBCC) and the International Building Code (IBC) now allow the use of encapsulated mass timber for taller structures, up to twelve storeys under the NBCC (2020) and eighteen storeys under the IBC (2021).

1.2 Research Needs

The heightened interest in sustainable construction has led to an increase in the usage of mass timber. The mechanical properties of mass timber make it appropriate for structural components in buildings and bridges. This has led to government measures to promote the mass timber sector development, in alignment with new building provisions that allow mass timber in taller structures. Despite recent advancements, more research is required to thoroughly understand and optimize the seismic performance of mass timber structures, especially those with braced timber frames (BTFs).

BTFs are an increasingly common Lateral Load Resisting System (LLRS) that combines timber diagonals, steel plates, and dowel-type fasteners. These connections are intended to yield and dissipate seismic energy while maintaining the structural integrity of timber elements. However, achieving sufficient lateral stiffness and ductility in bracing connections is essential. Poor detailing could result in brittle failure under relatively low loads (Popovski et al., 2002). Dowel-type connections are popular in BTFs because they are simple and code compliant, but system-level testing demonstrates that variations in timber qualities may lead to asymmetric brace end deformation, reducing their global drift capacity (Popovski et al., 2003). This highlights the significance of understanding the unsymmetrical deformation of connections in BTF cyclic loading, and modelling BTF behaviour as an equivalent nonlinear spring can be a valuable and accurate technique for capturing the ductility of the system (Chen & Popovski, 2020, 2021).

Canadian design standards do not provide clear ductility guidance on ductile BTF dowel-type connections and as a result, designers face uncertainty when designing connections for specific seismic performance objectives or insufficient ductility, which could impact the structure's seismic resilience. A possible option is to design connections as fuses, which are sacrificial elements to dissipate seismic energy, that can be quickly replaced if damaged (Bruneau & Reinhorn, 2007). In addition, self-centering connections using shape memory alloys (SMAs) may improve seismic resistance by reducing residual drifts.

SMAs exhibit superelastic behaviour, which allows it to experience large strains with minimum permanent deformation. Components made of SMA can be easily integrated into various structural connections, which have shown the ability to minimize peak interstorey drift while effectively eliminating residual drift. Although SMA research has primarily focused on its application for steel structures, recent studies have shown a similar response in timber structures. As a result, the system-level behaviour of SMA bolts for BTF structures should be evaluated to confirm the self-centering potential found at the connection level.

Advanced numerical modelling makes system-level analysis easily accessible, providing a method to simulate the seismic response of BTFs without extensive experimental work.

1.3 Objectives

This research aims to establish a numerical framework to evaluate the seismic performance of BTFs with SMA dowel-type connections in comparison with conventional steel fastener connections. Numerical models of BTFs are developed using connection-level models calibrated from experimental hysteresis behaviour, enabling detailed nonlinear time history analyses. The presented framework bridges the existing gap between experimental observations at the connection level and practical structural applications, specifically evaluating how the self-centering performance found at the connection-level translates to system-level behaviour of BTFs in moderate seismic zones in Canada.

To complete this numerical investigation, the following specific objectives were defined:

1. Develop a numerical framework in OpenSees that is able to capture the nonlinear behaviour of BTFs with steel and SMA dowel-type connections by:
 - a. Calibrating uniaxial materials for connection-level models based on experimentally-obtained hysteresis behaviour,
 - b. Defining brace-level models incorporating the concept of stronger and weaker connections
 - c. Constructing frame-level models to perform pushover and nonlinear time history analyses
2. Assess the self-centering ability of SMA dowel-type connections in BTF system-level models using the numerical framework developed in OpenSees
3. Compare the seismic performance of frames with SMA-connection against frames with traditional steel connections in BTF prototype buildings.

1.4 Scope

This research evaluates the structural performance of a representative BTF building designed for a site Class C location in Ottawa, which was selected as a representative location for moderate seismic conditions. Considering office occupancy, a typical column spacing of five metres and a consistent storey height of four metres were chosen to provide adequate ceiling clearances often required in office buildings. A post-and-beam system was selected as this system is commonly adopted for various occupancy with structural grids ranging between 4 and 6 metres (Kesik & Martin, 2021).

The post-and-beam system consists of a framework composed of glued laminated timber (glulam) columns and beams, which support roof and floor panels constructed using cross-laminated timber (CLT). Both glulam and CLT were selected, given that they are engineered wood products widely used in Ontario due to their availability. Glulam in Canada typically comes from two primary species groups: Douglas Fir-Larch and Spruce-Pine-Fir (SPF) (CWC, 2024a). SPF was selected for this study because it is abundantly available throughout Canada (CWC, 2024b). Regarding the bracing system, single diagonal glulam braces were used, as they are a suitable alternative for low- to mid-rise buildings. Moreover, this bracing configuration can be easily integrated into post-and-beam frames using uniaxial dowel-type connections, such as the experimental tests utilized to calibrate the uniaxial spring models.

1.5 Thesis Organization

The thesis is structured into seven chapters that build progressively from concept verification to system-level evaluation:

Chapter 1 provides a brief introduction to the research topic. This chapter presents the motivation for investigating resilient mass timber buildings and states the objectives, scope, and thesis outline.

Chapter 2 presents a detailed literature review covering the existing studies on wood material behaviour, dowel-type connections, braced timber frames, SMA properties, and numerical modelling of braced frames.

Chapter 3 describes the proof-of-concept framework to develop frame-level numerical models for BTFs from uniaxial material models calibrated using experimental data. The frame-level models are subject to pushover and time history analyses.

Chapter 4 introduces the design of the building prototypes, detailing gravity and lateral systems, and describing provisions to achieve ductile failure modes of the BTFs connections.

Chapter 5 describes the modelling process of the prototype building frames and the numerical analysis for nonlinear time history procedures, including the selection and scaling of ground motion records.

Chapter 6 presents and discusses the results obtained for both SMA- and steel- connected frames including periods, peak and residual interstorey drift, and peak floor acceleration.

Chapter 7 concludes the research by summarizing the key findings and highlighting the potential of using SMA dowel-type connections in BTFs.

CHAPTER 2. LITERATURE REVIEW

2.1 Overview

This chapter summarizes the fundamental knowledge regarding the structural behaviour of braced timber frames (BTFs), the research work conducted on the use of shape memory alloy (SMA) dowel-type connections, and the numerical modelling for BTFs. The literature review begins by looking at wood as an engineering material, including Engineering Wood Products (EWPs) given their growing use in modern constructions. Then, the chapter discusses the use of BTFs, specifically those with dowel-type connections and their seismic provisions. SMA connections are also investigated, emphasizing their mechanical behaviour and integration into BTFs to improve self-centering properties. Finally, the chapter looks at numerical modelling techniques with OpenSees, including both uniaxial spring models and frame-level models with SMA connections.

2.2 Wood Material

2.2.1 Mechanical Properties

Wood exhibits anisotropic behaviour due to its intricate cellular and fibrous composition, and is typically modelled as a three-dimensional orthotropic material, with distinct mechanical properties along its longitudinal (L), radial (R), and tangential (T) axes (Brémaud et al., 2011; Forest Products Laboratory, 2010; Hirai, 2005). Figure 2.1 shows the anisotropic directions of wood. For structural engineering purposes, these directions are generally simplified into two principal orientations: parallel-to-grain (longitudinal) and perpendicular-to-grain (radial and tangential). Mechanical performance is highly direction-dependent, with wood demonstrating significantly greater strength and stiffness when loaded parallel-to-grain than when loaded perpendicular-to-grain (Chen et al., 2022). For instance, clear softwoods at 12% moisture have tensile strengths of 70-140 MPa when loaded parallel-to-grain, but only 3-8% of such values when loaded perpendicularly (Harte, 2009). Similarly, compression parallel to grain reaches 30-60 MPa, whereas perpendicular compression only achieves 10-20% of that capacity (Harte, 2009).

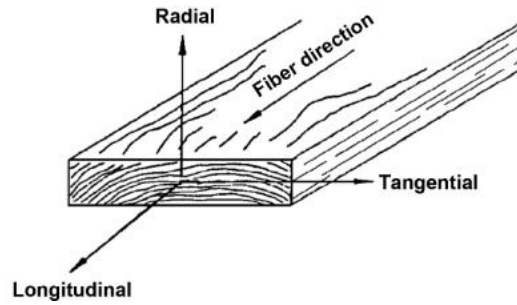


*Figure 2.1. Three Principal Axis of Wood
Retrieved from U.S. Dept. of Agriculture, USDA (2010)*

In addition, hygroscopic properties of wood and time-dependent behaviour introduce additional design complexities. Moisture within the cellular structure exists as free water in lumina and cavities and as bound water within the cell-wall matrix (Legg & Tingley, 2020; Ramage et al., 2017). The fibre-saturation point (FSP), approximately between 26% and 30% moisture content, marks the threshold above which further moisture does not affect the physical or mechanical characteristics of wood (USDA, 2010). Below the FSP, loss of bound water induces tangential shrinkage, leading to warping or internal stresses (Legg & Tingley, 2020). In contrast, moisture contents above the FSP reduce strength, requiring design reduction factors in humid or exterior environments, while prolonged wetting can promote decay (Legg & Tingley, 2020). Cyclic moisture fluctuations also produce dimensional changes that generate internal stresses, which must be accounted for in connection detailing. Finally, wood's viscoelastic nature leads to creep and fatigue under sustained loading: a constant bending load retains only 90% of its initial capacity after one hour and may decline below 65% over several decades (Breyer et al., 2020; Forest Products Laboratory, 2010).

2.2.2 Structural Behaviour

Under tensile loading parallel-to-grain, timber responds in a linear-elastic behaviour until brittle rupture of the grains causes failure (Buchanan, 1990). Tension perpendicular-to-grain also fails in a brittle manner, and its significantly reduced capacity (in comparison with tension parallel-to-grain) limits its use in certain applications (CSA, 2019; Chen et al., 2022).

When loaded in compression parallel-to-grain, wood has a bilinear stress-strain response, an elastic segment up to the proportionate limit, followed by a reduced-stiffness branch after yielding. Failure in compression parallel-to-grain loading can occur through crushing, wedge splitting, shearing, splitting, end rolling, or brooming, as well as combined crushing and parallel-to-grain shearing, as shown in Figure 2.2 (ASTM, 2022). Initial theoretical models, including Neely's elastoplastic formulation (1898) and Glos's

falling-branch model (1978), were eventually combined into the commonly used bilinear model by Bazan (1980) and Buchanan (1990), illustrated in Figure 2.3.

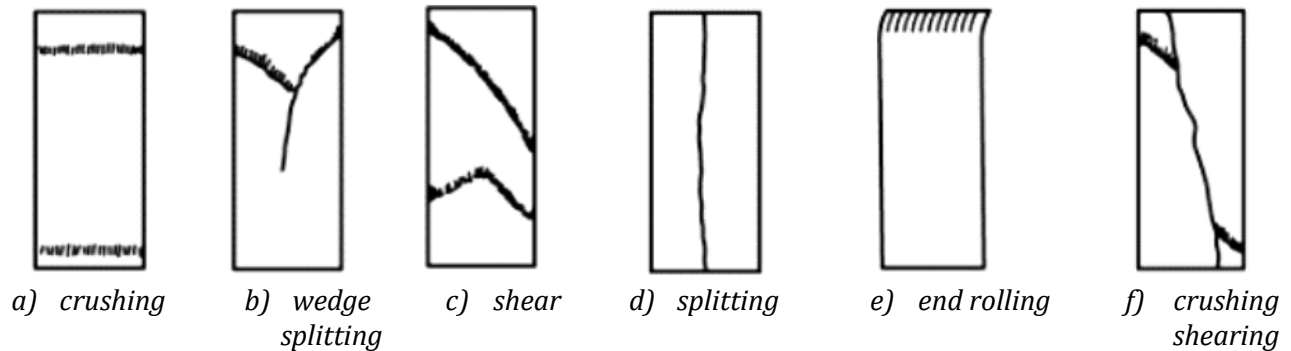


Figure 2.2. Compression Failure Modes - Parallel-to-Grain
Adapted from (ASTM, 2022)

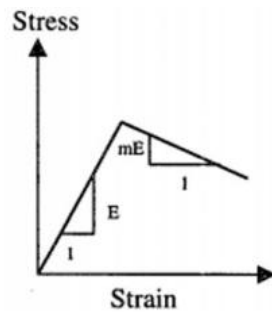
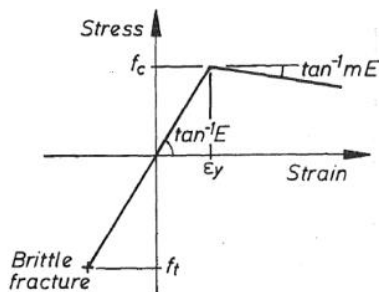
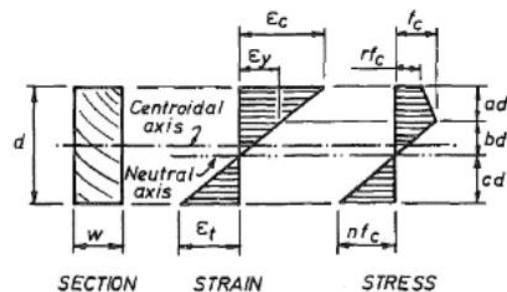


Figure 2.3. Stress-Strain Model for Wood Compression of Bazan (1980) and Buchanan (1990)

Wood beams in flexure engage both tensile and compressive grain-parallel behaviour. Buchanan's (1990) model (Figure 2.4a) uses a linear-elastic tension that fails in a brittle manner at f_t and a bilinear compression curve that yields at stress f_c and strain ϵ_y , then the capacity reduces as the fibres crush. Figure 2.4b shows the resulting section stress-strain profile. In practical applications, it is important to check perpendicular-to-grain compression at bearings and tensile capacity at notches or splits to avoid localized failure.



a) axial loading



b) flexural loading

Figure 2.4. Strength Models of Wood
Retrieved from Buchanan (1990)

Bending strength in clear wood members is determined by tension failure at the extreme tension fibre, which usually occurs where bending moments peak. Gradual tensile failure causes several splinters to form from the outer fiber (Figure 2.5c), whereas brash failure (Figure 2.5d) results in a rapid break with little or no splintering (ASTM, 2021a). Compressive failure on the opposing face is seen as localized fiber crumpling or wrinkling (Figure 2.5e). Knots present high-stress concentrations that induce fractures (Figure 2.5b), whereas existing splits or checks can develop into longitudinal shear failures (Figure 2.5f). These phenomena work together to define the limit states for the flexural design of timber elements.

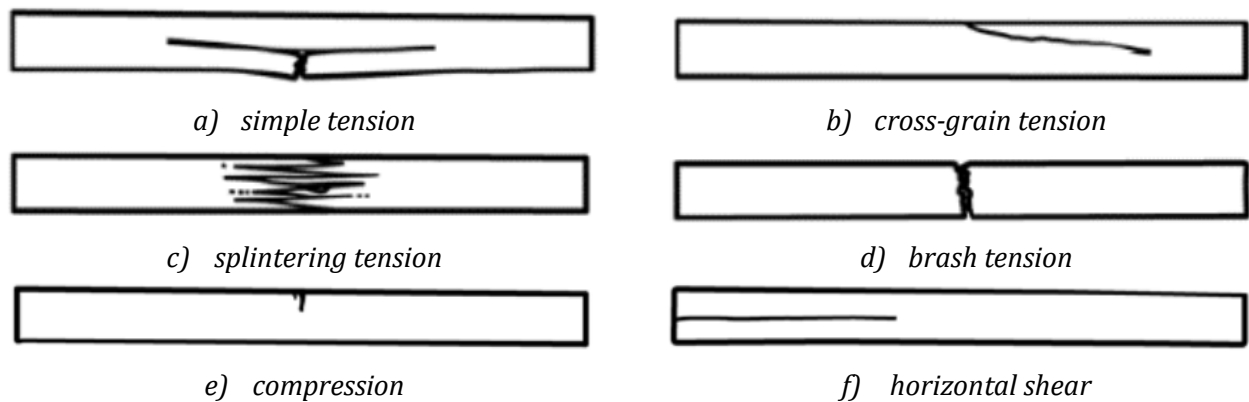


Figure 2.5. Failure Modes - Wood Flexure
Retrieved from ASTM (2022)

2.2.3 Engineering Wood Products

Engineering wood products (EWPs) including mass timber products, are fabricated by joining small wood pieces (e.g., dimensional lumber, veneers, strands) with adhesives, dowels, nails, or screws to form large structural members (Gong, 2022). Figure 2.6 presents examples of EWPs manufactured from dimensional lumber, including glued-laminated timber (glulam), cross-laminated timber (CLT), dowel-laminated timber (DLT), and nail-laminated timber (NLT). Figure 2.6 also shows structural composite timber products fabricated from veneers or strands such as parallel strand lumber (PSL), laminated veneer lumber (LVL), laminated strand lumber (LSL), and mass plywood timber (MPP). Mass timber's popularity increases the ability to construct taller timber buildings, with rapid assembly, great strength, environmental benefits, and an aesthetically pleasing look (Natural Resources Canada, 2021). Mass timber is stronger than the individual pieces used in the manufacturing process, has less variability than dimensional or heavy timber, and come in various sizes that are not limited by log diameter (Gong, 2022). In glulam, all laminates align with the member longitudinal axis; in CLT, laminates alternate at right angles to make panels suitable for walls and floors (Natural Resources Canada, 2021).

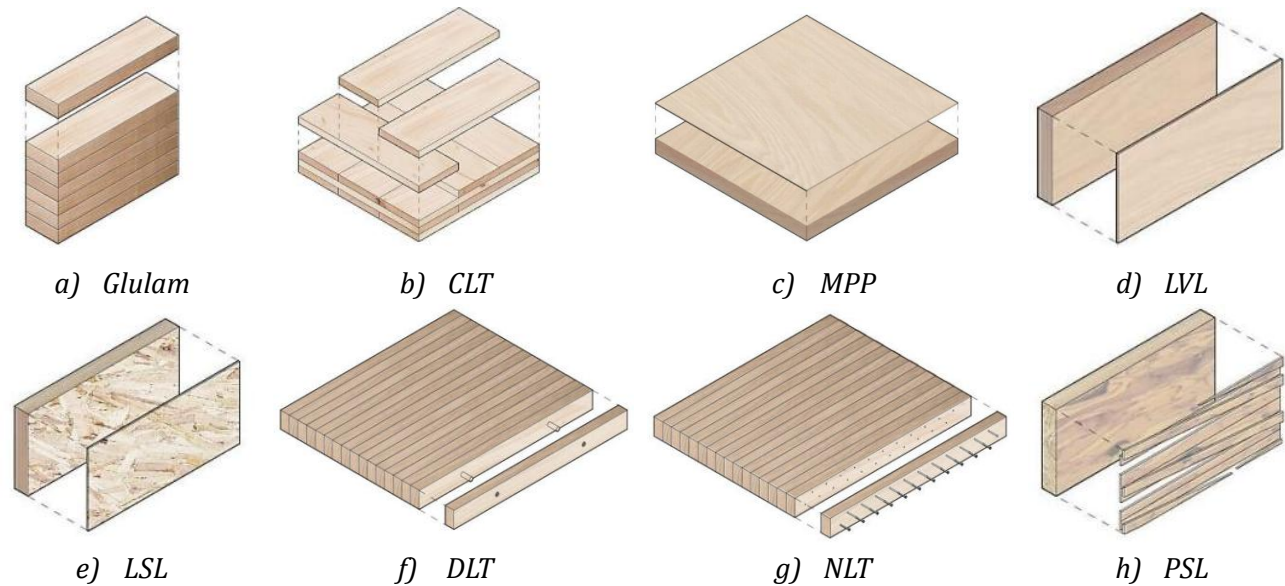


Figure 2.6. EWPs Examples
Retrieved from Patel, S./Mass Timber Institute (2020)

2.2.4 Variability and Defects

The structural performance of wood varies naturally between softwoods (conifers) and hardwoods (angiosperms), with CSA O86 classifying common construction species into groups such as Spruce-Pine-Fir (SPF), Hem-Fir, and Douglas Fir-Larch (CSA, 2019). Even within a single species, environmental factors such as temperature, precipitation, soil, and light produce notable variations in knot frequency, density, and ring shape (Ramage et al., 2017). To handle this variability, timber is usually visually graded to identify obvious faults. Other common grading methods are the machine stress-rated (MSR) or machine-evaluated (MEL) to further classify wood based on defect criteria. MEL differs from MSR by mandating daily quality assessments for tensile capacity, flexural strength, and stiffness (NLGA, 2019).

2.3 Braced Timber Frames

2.3.1 Introduction

The fast adoption of mass timber construction in Canada brought attention to the need for lateral-load resisting systems (LLRS) that can mitigate wind and seismic forces (Popovski et al., 1999). In response, braced timber frames (BTFs), which consist of rectangular portal frames with diagonal braces on each level, have come up as a common alternative. BTFs combine the high axial stiffness of structural timber diagonals (typically glulam) with slender steel plates and dowel-type fasteners at brace ends. Under the capacity-based design framework in NBCC (2020), these connections are designed to yield, therefore distributing

seismic energy, prior to wood crushing or fracture, conserving the elastic integrity of the primary timber elements for efficient post-event repair (NBCC, 2020; Popovski et al., 1999).

Designing an efficient BTF requires achieving two key performance objectives. First, the bracing connections should provide enough lateral stiffness to keep interstorey drift within code limitations. Second, they need to have sufficient ductility to absorb seismic energy without causing brittle timber failure. Inadequate detailing can cause stress perpendicular to the grain, leading to sudden splitting under relatively small loads (Popovski et al., 2002). Although specialized connectors (e.g., split rings, shear plates, or glued in rods) are more novel alternatives, dowel-type fasteners remain popular in BTFs because of their simple detailing, well-understood behaviour, and compliance with Canadian standards.

Early system-level testing highlights the importance of connection behaviour in BTFs. The shake table studies by Popovski et al. (2003) shown in Figure 2.7 explain that even with symmetric detailing, the natural variability in timber properties can cause asymmetric brace end deformation. This results in one connection yielding prematurely; thus, this connection defines the system drift capacity. Popovski et al. (2003) described this phenomenon by stating that the deformation capacity of the brace is not double that of a single connection measured individually. The connection that undergoes larger nonlinear deformations is named the "weaker" connection, while the other one is the "stronger" connection (Popovski, 2000; Popovski et al., 2003).

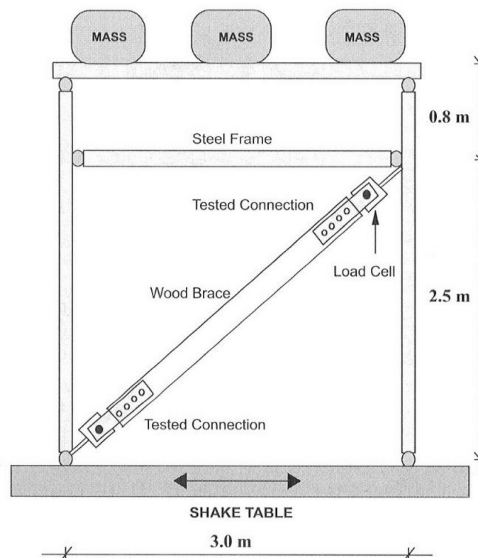


*Figure 2.7. Shake Table Test Schematic
Retrieved from Popovski et al. (2003)*

The findings from experimental testing program of brace members with timber rivet connections by Popovski (2004) confirmed the concept of the stronger and weaker connections. These experimental results (shown in Figure 2.8) depict the difference in deformation recorded at each brace end. More recent experimental results also show this difference in the displacement between the top and bottom connections (Baird et al., 2024; Chen et al., 2019). Therefore, understanding the concept of stronger and weaker connections is essential for understanding how BTFs behave seismically. The behaviour of BTFs can be accurately modelled as one stronger connection, one weaker connection, and the elastic brace as an equivalent nonlinear spring (Chen & Popovski, 2020, 2021). This method considers that one connection is likely to experience larger deformation and energy dissipation than the other connections during seismic events (Chen & Popovski, 2021).

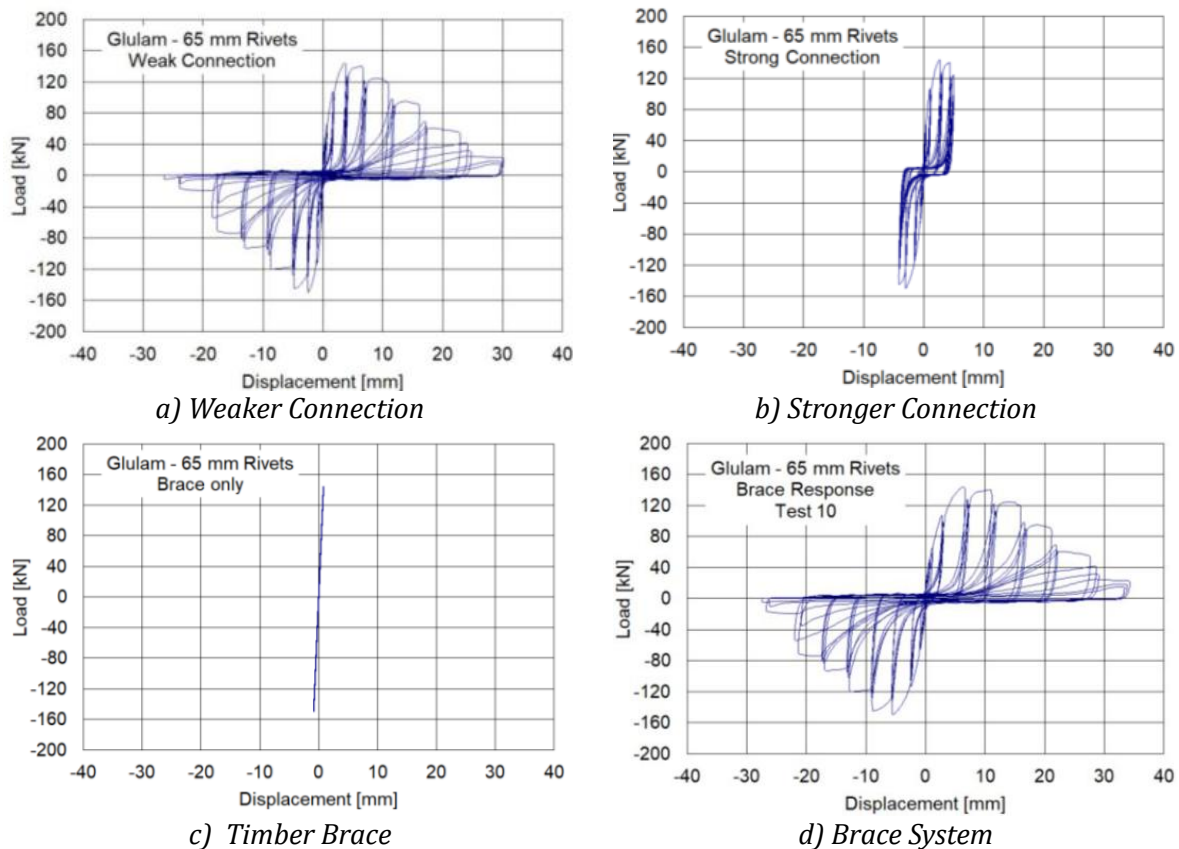


Figure 2.8. Experimental Hysteresis Curves of BTFs with Riveted Connections
Retrieved from Popovski (2004)

Chen and Popovski (2021) used the test results of a glulam brace with riveted connections tested by Popovski (2004) as a benchmark to determine an equivalent nonlinear element in a numerical study of braced wood frame buildings. This method can capture the overall response of the BTF system while requiring little numerical processing resources and complexity (Chen & Popovski, 2020). In addition, Chen

et al. (2019) tested diagonal braces with bolted connections under monotonic and cyclic loading protocols (Figure 2.9), and discovered that the top and bottom connections deform differently, reinforcing the concept of stronger and weaker connections.

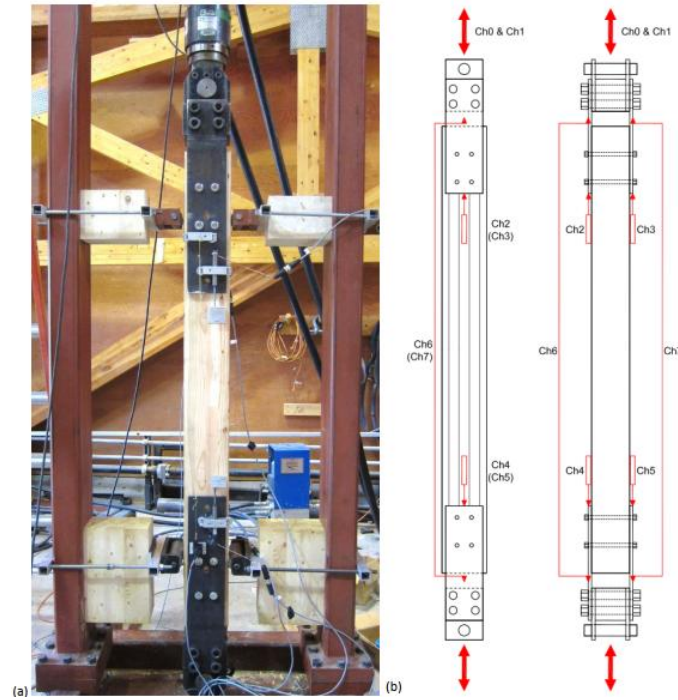


*Figure 2.9. Test Setup for Brace-Level Experimental Testing
Retrieved from Chen et al. (2019)*

The study by Chen and Popovski (2020) derived a relationship between connection ductility and system ductility in BTFs as the stiffness ratio between the connection and the diagonal bracing, as well as the number of stories, all influence the system ductility. The equations were derived considering the concept of stronger and weaker connections at the brace ends. The authors validated their proposed relationship by performing pushover analysis on numerical building prototypes, incorporating equivalent nonlinear springs as timber braces using the weaker and stronger connection principles (Chen & Popovski, 2020). The results of the validation between the numerical and theoretical relationship methods showed an agreement within a 5% error when determining the ductility of the system. Overall, modelling BTF behaviour with one stronger connection, one weaker connection, and an elastic brace provides a practical and accurate method for capturing the system's ductility (Chen & Popovski, 2020; Popovski, 2000, 2004; Popovski et al., 2003).

2.3.2 Dowel-Type Connections

Johansen's Yield Model (Johansen, 1949), also known as the European Yield Model (EYM), estimates the plastic capacity of dowelled connections by superimposing the bending resistance of steel dowels and the wood's embedment strength. Expressions developed for lateral capacity under both single and double hinge plastic mechanisms serve as the basis of European design criteria. The EYM modes for double-shear connections are provided in Figure 2.10. The fracture mechanics model presented by Jorissen (1998) adds to this by computing stress intensity around slotted bolt-holes as a function of dowel diameter, spacing, and edge distance, which are particularly important to address brittle failure modes, allowing for a more adequate analysis of connection behaviour.

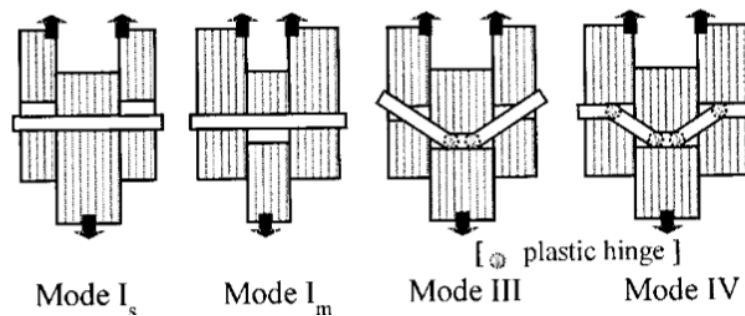


*Figure 2.10. European Yield Model models for Double Shear Joints
Retrieved from Smith and Foliente (2002)*

In Canada, the CSA O86 (2019) guidelines for dowel-type connections were developed using the EYM. Regarding the brittle failure modes, these are shown in Figure 2.11 and include row shear (shear forces), net tension (tension forces parallel to grain), group tear-out (a combination of shear and tension parallel to grain), and splitting (tension perpendicular to grain). These brittle failures can be prevented by proportioning adequate fastener spacing and end distance to edge in both parallel- and perpendicular-to-grain directions. However, to achieve greater ductility and energy dissipation capacity, it is ideal that connections fail in a ductile manner. The CSA O86 (2019) also provides guidelines to determine the lateral resistance of dowel-type connections (Table 2.1). The design of dowel-type connections is based on the lowest strength calculated from the ductile and brittle failure modes. However, connections governed by wood embedment strength failure modes (a, b, c, and f) can result in high stress concentrations, increasing the likelihood of brittle failures. In contrast, connections with failure modes combining wood embedment strength and fastener yielding (d, e, and g) are preferred as they behave in a ductile manner. Mode g, defined by two plastic hinges at the wood-steel interface, produces the most stable and energy-dissipating hysteresis loops. This mode is known as “Mode IV” as per the EYM.

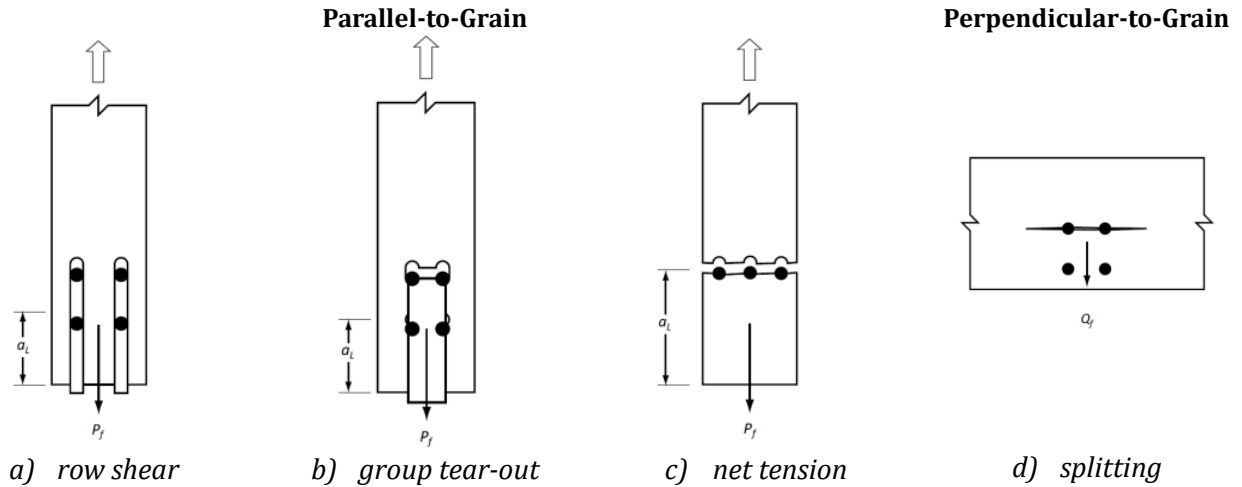


Figure 2.11. Brittle Failure Modes in Dowel-Type Connections
Reproduced from CSA 086 (2019)

Table 2.1. Lateral Yielding Resistance
Reproduced from CSA 086 (2019)

Mode	Design Equation	Single-Shear	Double-Shear
a)	$f_1 d_F t_1$		
b)	$f_2 d_F t_2$		
c)	$\frac{1}{2} f_2 d_F t_1$		
d)	$f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1+f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$		
e)	$f_1 d_F^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1+f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$		
f)	$f_1 d_F^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$		
g)	$f_1 d_F^2 \sqrt{\frac{2}{3} \frac{f_2}{(f_1+f_2)} \frac{f_y}{f_1}}$		

Subsequent studies further developed knowledge in dowel-type connections. Popovski et al. (2002) compared 65 mm timber rivets with 9.5 mm and 12.7 mm bolts in glulam under monotonic and cyclic loading. They found that rivets dissipate approximately 20% more energy, while slender bolts recover

comparable ductility from progressive wood crushing (Popovski et al., 2002). Baird et al. (2024) evaluated one plate versus two plate configurations using cyclic protocols, demonstrating that post-yield hardening results in stable, pinched loops, whereas softening causes uneven deformation between top and bottom fasteners and reduces global ductility.

Furthermore, self-tapping screws (STS) used perpendicular to the grain for reinforcement have proven to be quite effective to increase the ductility of the connections. Bejtka & Blaß (2005) found that STS reinforcement prevent splitting by achieving dowel-yielding failure modes. The authors report that connections with STS reinforcement increase their capacity up to 80% and 120% in connections with ductile- and brittle-failure behaviour, respectively. Similarly, Breijinck et al. (2023) examined joints proportioned to exhibit comparable dowel-yield and row-shear strengths and showed that adding STS reinforcement still produced significant increments in ductility and energy dissipation. They also observed that these gains became more pronounced as the slenderness (dowel diameter to timber thickness) ratio increased. In addition, Viau & Doudak (2021) and McGrath & Doudak (2021) demonstrated that under blast load conditions in shock tube tests, STS reinforcement also shifts failure from brittle splitting to ductile bolt yielding and wood crushing, substantially enhancing energy dissipation under high-rate loading.

Finally, Dorn et al. (2013) found that the EYM is conservative for medium to high slenderness (λ) dowels, but overpredicts capacity for $\lambda < 10$. They mentioned that roughening the dowel surface and maintaining λ greater than 10 to 12 ensures mode g failure. Sawata et al. (2006) independently verified shear strength calculations across timber thicknesses and dowel diameters, whereas Awaludin et al. (2008) confirmed that bolt pretension can only increase connection capacity by 5-10%, which is significantly less than linear elastic estimates.

2.3.3 Seismic Provisions

The NBCC (2020) includes ductility-based force modification factors (R_d), which reflect the energy dissipation and inelastic deformation requirements of various SFRS. For BTFs, these have been assigned a $R_d = 1.5$ for limited ductility and $R_d = 2.0$ for moderate ductility, with maximum height limits of 15 metres (SC3) and 20 metres (SC4), respectively (NBCC, 2020). These R_d values are nearly entirely derived from connection behaviour rather than timber member ductility. Although the NBCC refers to CSA O86 for connection design, this standard does not provide specific recommendations for which connector types and configurations meet limited or moderate ductility criteria. Analytical time history studies (Mitchell et al., 2003; Popovski & Karacabeyli, 2008; Popovski, 2009) proposed $R_d = 1.5$ for slender bolted connections and $R_d = 2.0$ for riveted connections. These values were confirmed by using a collapse margin ratio assessment

of thirty riveted archetype structures, which provides acceptable seismic safety margins under probabilistic conditions (Chen & Popovski, 2021).

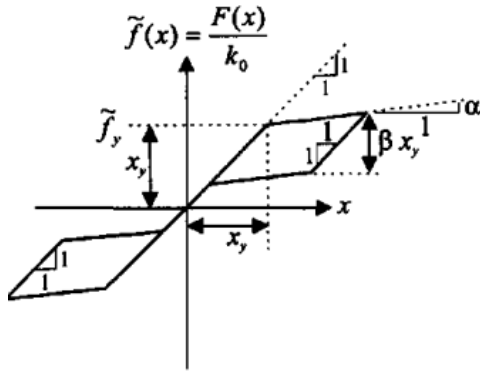
Chen & Popovski (2020) provided design guidelines defining the required connection ductility to achieve moderate and limited ductility levels as per the NBCC (2020), corresponding to R_d factors of 2.0 and 1.5, respectively. As seen in Table 2.2, the minimal connection ductility (ratio of ultimate displacement and yield displacement) for the moderately ductile category of BTFs is 11.5 if just one end of the diagonal brace yields, and 6.3 if both ends yield. For frames with low ductility, the minimum connection ductility corresponds to 5.4 with yielding in a single connection and 3.2 with yielding in both connections. This is important for seismic resilience in mass-timber buildings because it allows for a direct and quantitative correlation between the ductility of both end connections and the ductility of the system.

*Table 2.2 Ductility Requirements for BTFs
Retrieved from Chen and Popovski (2020)*

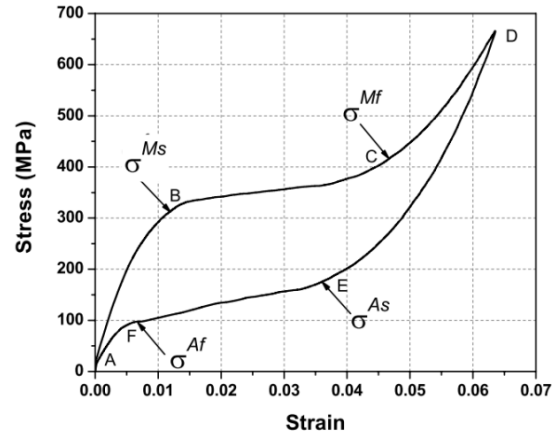
Category	R_d	Brace Ductility	Connection Ductility for Yielding in	
			Single Connection	Single Connection
Moderate Ductility	2.0	2.5	11.5	6.3
Limited Ductility	1.5	1.6	5.4	3.2

The CSA O86 currently lacks prescriptive ductility standards for brace-end dowel-type connections, and the NBCC Earthquake Design Subcommittee has warned that BTFs may be deleted from future code cycles if this gap continues (Chen & Popovski, 2020). To improve seismic resilience in mass-timber buildings, designers must detail these connections to sustain Mode “g” yielding, reinforce them against brittle failure, and validate their performance through cyclic testing. This can be achieved if the brace connections are designed as fuses, non-critical elements that absorb seismic energy but can be replaced rapidly if damaged (Bruneau & Reinhorn, 2007). This idea, when applied correctly, enables the quick replacement of connections and the preservation of large structural components. To perform as intended, such fuses must be located within the SFRS, allowing the structure to release energy while the gravity frame remains elastic (Farsangi et al., 2019).

Further research into fuses has led to current studies investigating structural connections with self-centering capacity, which reduce residual drifts to levels that eliminate the need for component replacement. To achieve this performance, a flag-shaped hysteresis is required in which the force-displacement curve crosses the origin upon reverse loading or unloading, as shown in Figure 2.12a. Such behaviour can be achieved by taking advantage of SMA superelastic properties (Figure 2.12b), which have inherent recentring capabilities that exhibit flag-shaped loops behaviour depending on the application.



a) *Flag-Shaped Hysteresis*
Retrieved from Christopoulos (2004)



b) *SMA Superelastic Behaviour*
Retrieved from Lagoudas (2008)

Figure 2.12. Self-Centering Behaviour

2.4 Shape Memory Alloy Connections

2.4.1 Mechanical Behaviour

Shape memory alloys (SMAs) are smart metals that can be “trained” to return to a preset shape. When they are bent, stretched, or twisted in one state, a change in temperature or unloading triggers an internal, solid-state phase switch that drives them back to their original form (Lagoudas, 2008). This built-in shape recall makes SMAs useful in many engineering and everyday products. In technical terms, SMAs exhibit a reversible and diffusionless phase transformation between a high-temperature, austenite, and a low-temperature, martensite (Lagoudas, 2008).

The earliest observations of reverse martensitic transformation in Cu–Zn and Cu–Al alloys were reported by Kurdjumov and Khandros (1949), although they did not investigated the reverse transformation of martensite. It was not until the identification of the Nickel-Titanium (NiTi) alloy during heat-shield materials research that the reverse transformation of martensite became fully characterized (Lagoudas, 2008). Since its discovery, NiTi, also known as Nitinol, has emerged as the typical SMA, owing to its pronounced superelastic response and excellent corrosion resistance. These properties have driven extensive adoption of NiTi in biomedical devices, such as sutures, stents, and orthodontic archwires, aerospace components, and consumer products like eyeglass frames (Lecce & Concilio, 2015). More recently, the unique practical abilities of SMAs have driven their integration into structural engineering applications, where they offer new opportunities for enhancing the resilience and serviceability of civil infrastructure (Lecce & Concilio, 2015).

As shown in Figure 2.13, SMAs display the shape-memory effect (SME) within the martensitic regime ($T < A_f$), in which stress-induced detwinning of the twinned martensite can be fully reversed by heating above the austenite finish temperature (A_f) (Lagoudas, 2008). Unloading leaves a residual strain in the detwinned martensite, which reverts to its original twinned configuration upon heating above A_f and subsequent cooling below the austenite start temperature (A_s) (Lagoudas, 2008). This behaviour shows that under increasing stress, twinned martensite de-twins at the start stress (σ_s), creating a plateau until the stress level (σ_f), when full detwinning occurs (Lagoudas, 2008). At temperatures over A_f (Point F), austenitic SMAs exhibit super elasticity (SE), a stress-induced transformation to martensite under load follows a flag-shaped hysteresis loop and fully recovers upon unloading, producing recoverable strains of 6-8% (Lagoudas, 2008; Seo et al., 2015).

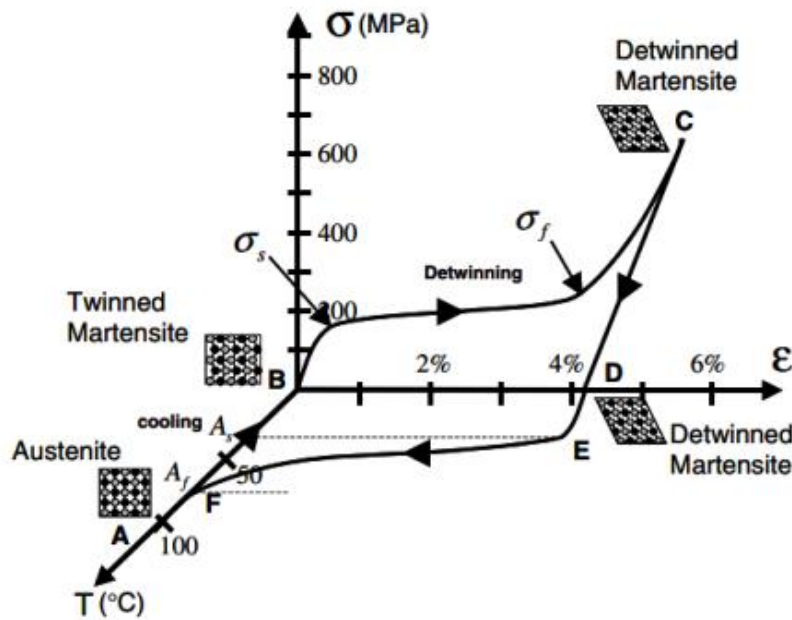


Figure 2.13. SMA Stress-Strain-Temperature Plot at SE
Retrieved from Lagoudas (2008)

Seo et al. (2015) described that loading an SMA induces a stress-driven martensitic transformation, from the martensite-start stress (σ_{Ms}) to the martensite-finish stress (σ_{Mf}), and that upon unloading the alloy retraces its path back to austenite, recovering strain as it passes through the austenite-start (σ_{As}) and austenite-finish (σ_{Af}) stresses. Figure 2.14 overlays these SME and SE responses to illustrate how the mechanical behaviour of the material shifts with its transformation temperatures. While Figure 2.15 illustrates these effects separately to highlight their differences. At temperatures below M_f , the stress-strain loop follows the profile in Figure 2.15a, resulting in significant residual strain upon unloading (Tzou & Baz,

2001). Increasing the temperature to A_f allows for full recovery of deformation, resulting in the shape-memory loop seen in Figure 2.15b (Tzou & Baz, 2001). In this study, the SE behaviour of SMAs at the austenite phase is relevant to evaluate the self-healing ability of dowels or bolts after unloading. In contrast, the heat treatment for reaching the self-centering effect would not be feasible in dowel-type connections.

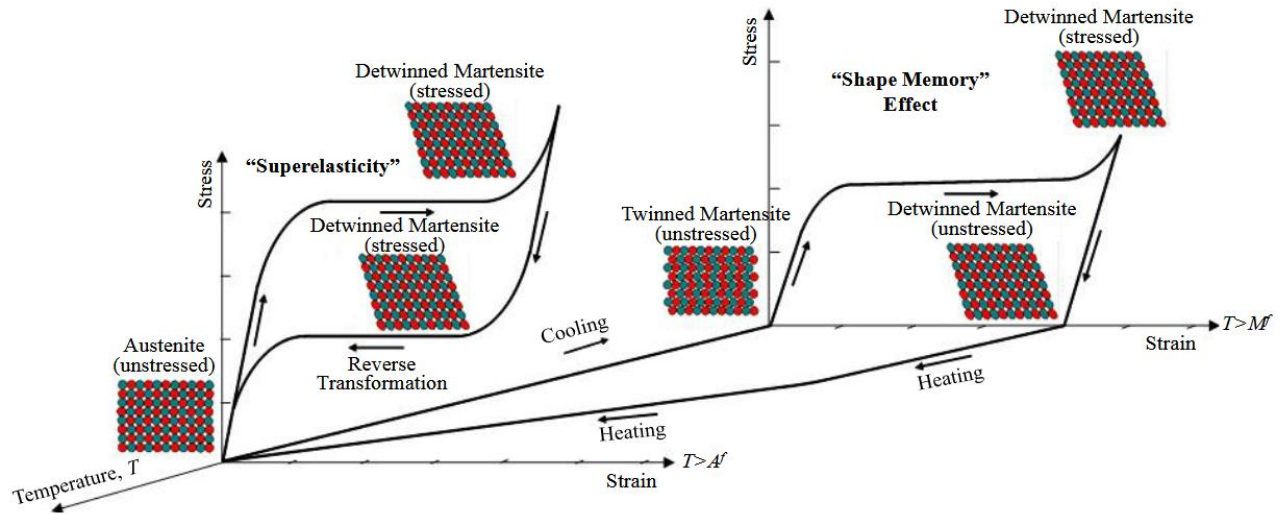


Figure 2.14. SMA Stress-Strain-Temperature Phases
Retrieved from Seo et al. (2015)

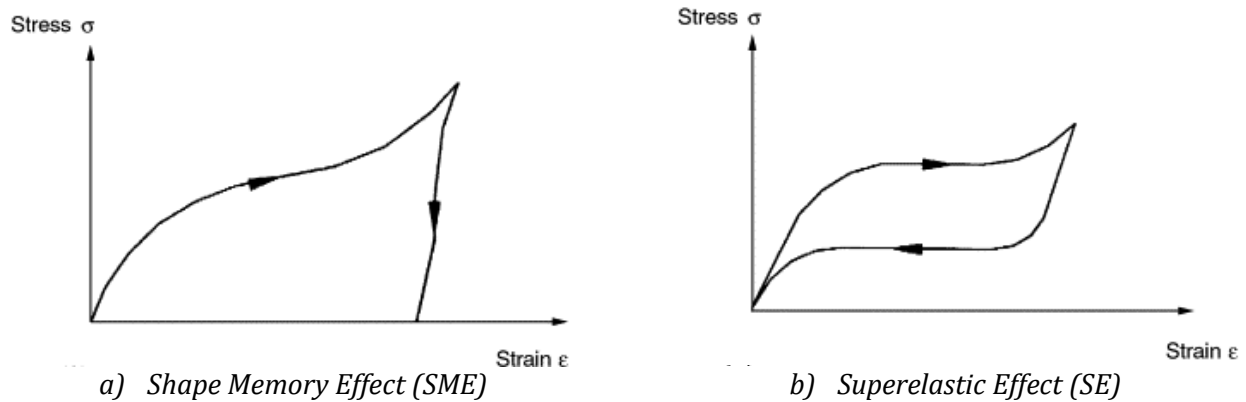


Figure 2.15. SMA Stress-Strain Characteristics
Retrieved from Tzou and Baz (2001)

2.4.2 Applications of SMAs in Braced Frames

SMAs have been investigated in a variety of bracing and connection systems because of their ability to withstand large reversible strains, which minimizes residual deformation following earthquakes and reduces post-event repair costs (McCormick et al., 2007). Over the last two decades, research has shown

that SMA elements may be used at a variety of structural scales, from individual bolts and dowels to full-length bracing and dampers, each offering different advantages over traditional steel components.

Connection-Level Devices

Early efforts focused on SMA bolts or pins in steel frames. Fang et al. (2015) demonstrated that NiTi rods provide excellent axial recentering during cyclic tension testing, whereas large-diameter bars waste less energy and accumulate more residual strains under shear. Ma et al. (2007) improved this concept by replacing all carbon-steel bolts in a beam-to-column end-plate joint (Figure 2.16) with NiTi fasteners, resulting in a "super-elastic hinge" that recovered nearly all imposed deformations. However, shear transfer to the column flange became a design consideration. A hybrid arrangement that combines superelastic bolts with conventional high-strength bolts (Figure 2.17) was presented by Fang et al. (2015). These connections preserved the initial stiffness of the connection and enhanced recentering capacity, but cyclic tests revealed thread-root failures in the SMA bolts, emphasizing the need for detailed bolt-region design (Fang et al., 2015).

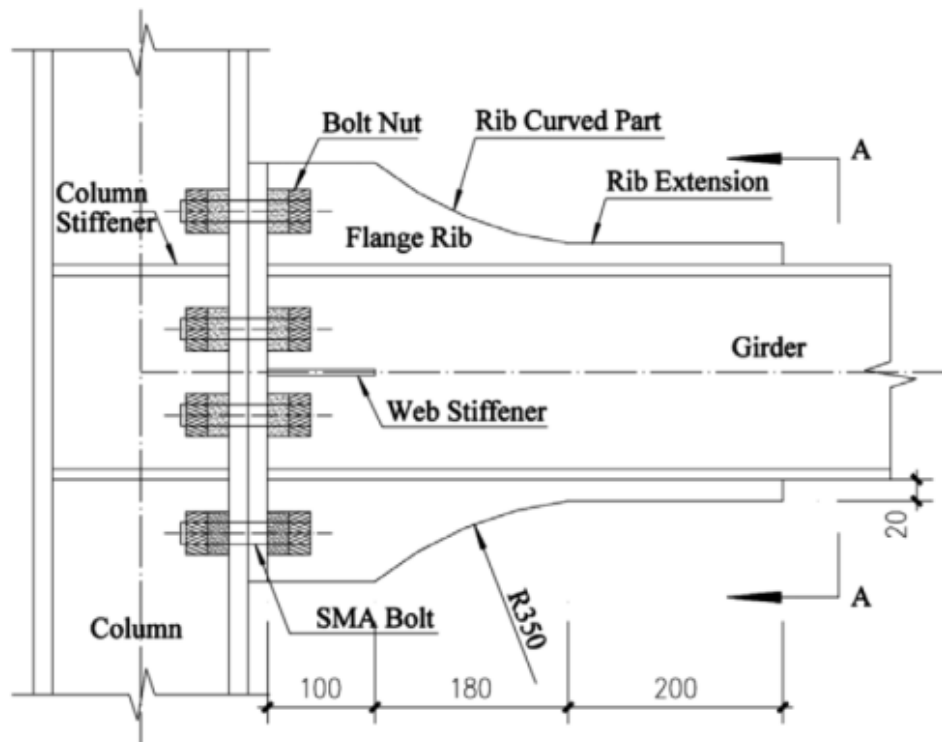


Figure 2.16. SMA Bolts Connection at Beam-to-Column
Retrieved from Ma et al. (2007)

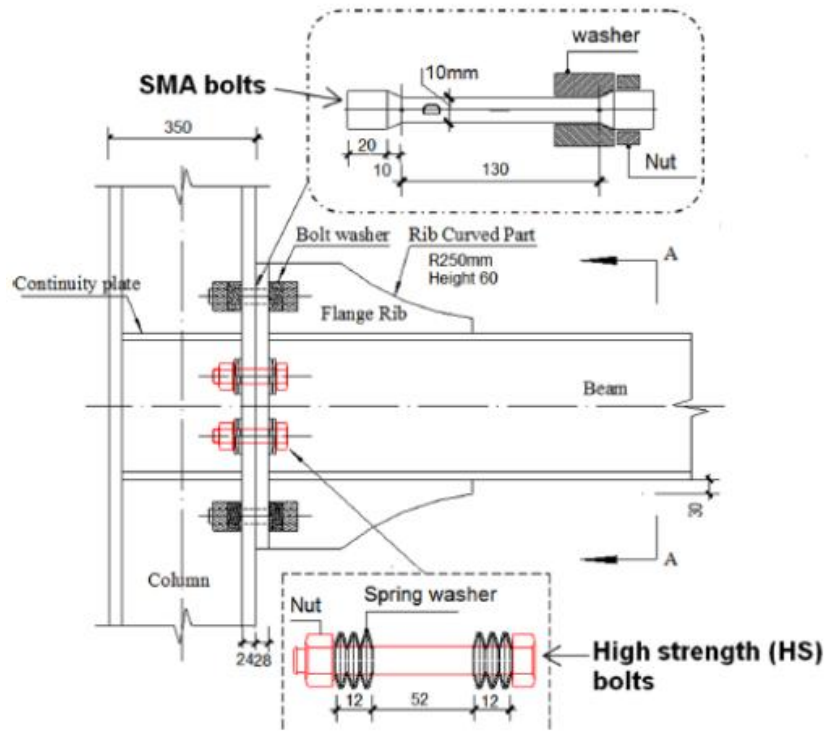


Figure 2.17. Superelastic Beam-to-Column Connection
Retrieved from Fang et al. (2015)

Based on these findings, self-centering energy-dissipative beam-to-column connections were validated using shake-table testing (Erochko et al., 2013) and full-scale tests (Christopoulos et al., 2008). These experiments revealed that SMA-based connections effectively reduce peak interstorey drift while essentially eliminating residual drift. However, the lower energy dissipation compared to steel connections is a design compromise that needs to be handled through hybridization or supplementary damping.

Brace-Level Steel Frames

Several studies have incorporated SMA elements directly into bracing members, combining flag-shaped hysteresis with increased energy dissipation. Zhang et al. (2022) created the SMA Viscoelastic Hybrid Bracing (SCVEB) unit, in which NiTi cables implanted in a central steel tube work in combination with viscoelastic dampers (Figure 2.18). In this experiment, cyclic tests resulted in stable flag-shaped loops and reduced peak interstorey drift by 35% compared to frames with SMA-only braces. Similarly, Miller et al. (2012) designed a self-centering buckling-restrained brace (BRB) by post-tensioning a standard steel core with NiTi rods (Figure 2.19), resulting in strong energy dissipation and little residual drift. Qiu and Zhu (2017) used a quarter-scale, two-storey steel braced frame to incorporate superelastic NiTi wires as shown

in Figure 2.20. The SMA-reinforced frame was assessed experimentally and numerically to withstand many simulated earthquakes, and showed minimum damage and permanent deformation of the frame (Qiu & Zhu, 2017).

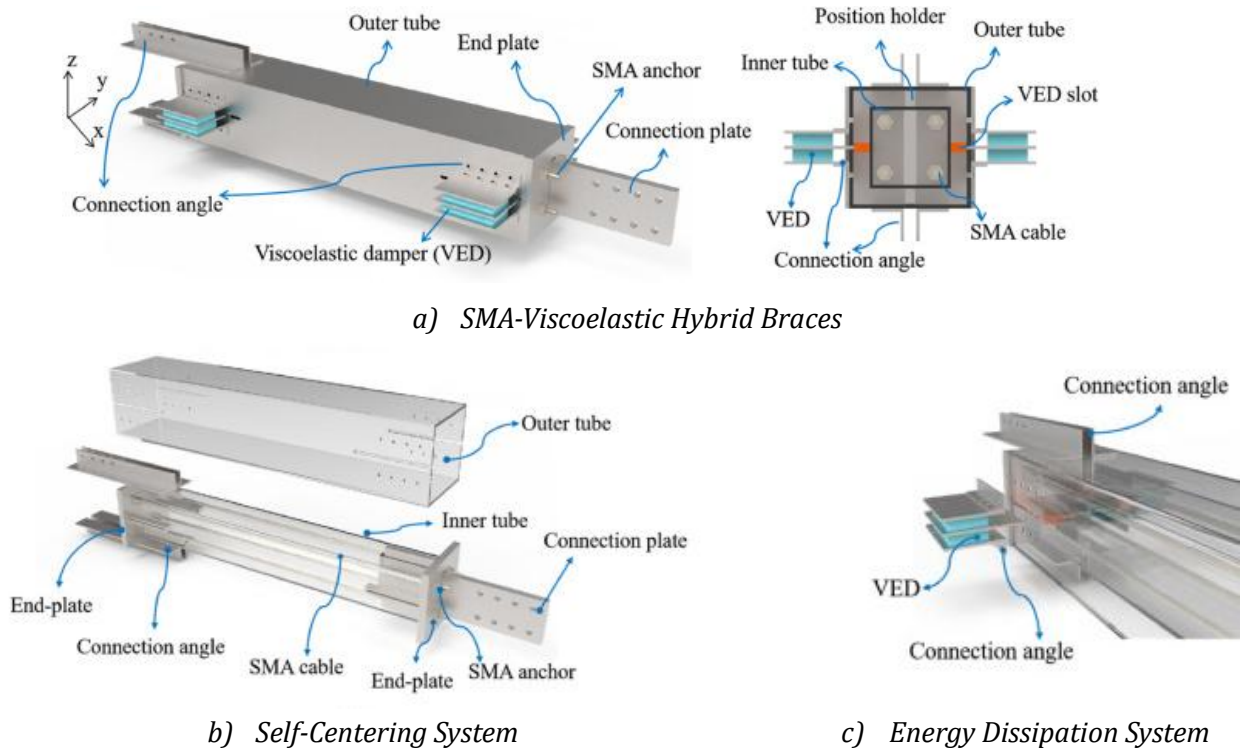


Figure 2.18. Schematics for SMA-Viscoelastic Hybrid Braces System
Retrieved from Zhang et al. (2022)

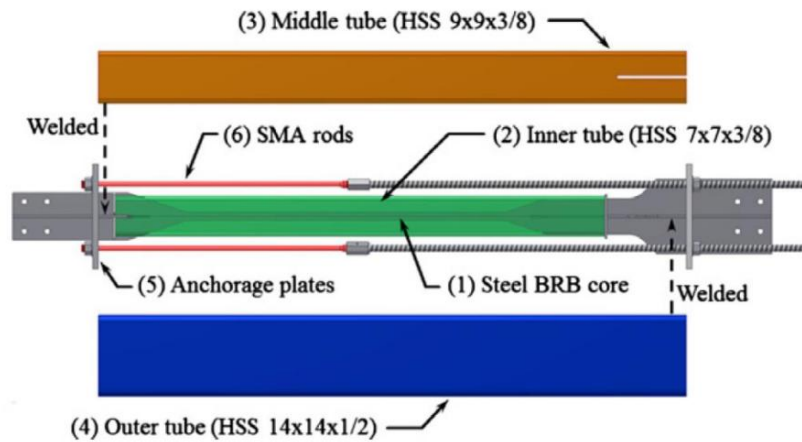


Figure 2.19. Self-Centering BRB Testing Schematic
Retrieved from Miller et al. (2012)

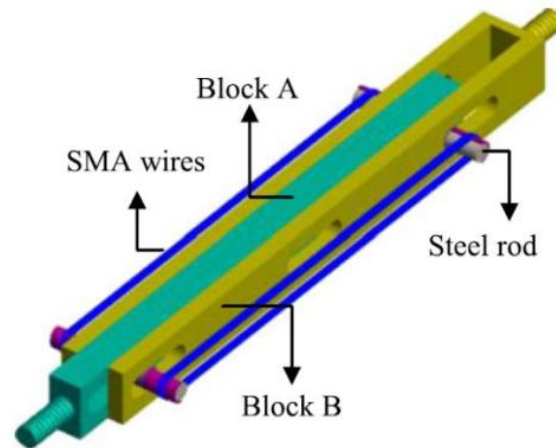
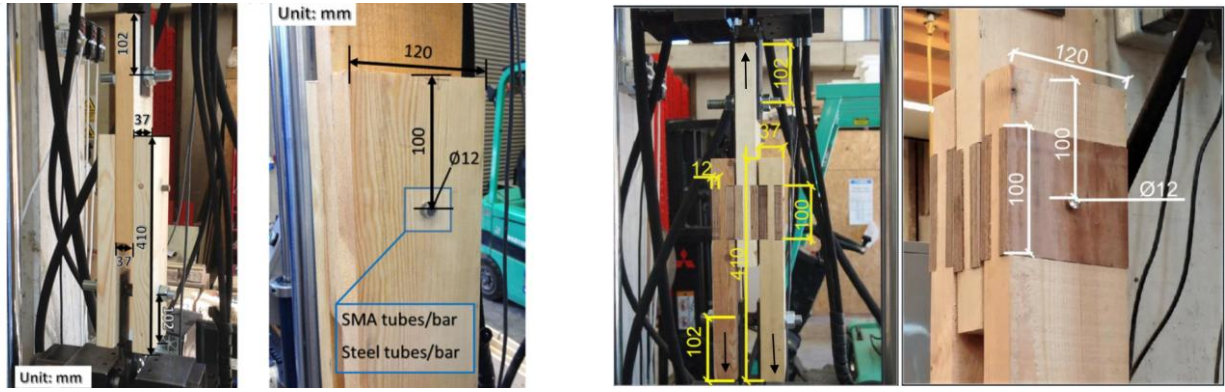


*Figure 2.20. Self-Centering Steel Brace with NiTi Wires
Retrieved from Qiu and Zhu (2017)*

The findings from these experimental tests were confirmed by numerical studies. McCormick et al. (2007) used nonlinear time-history studies to show that SMA braces minimize peak and residual interstorey drifts in three- and six-storey concentrically braced frames. Asgarian and Moradi (2011) examined diagonal, split-X, and chevron layouts and concluded that SMA braces reduce residual roof displacement and peak drift more effectively than buckling-restrained bracing. Response-modification factors for SMA-braced frames have been also calculated for various heights and bay numbers (Ghassemieh & Kargarmoakhar, 2013). In addition, to optimize self-centering performance, optimal placement studies advise positioning SMA segments inside gusset plates or at brace ends (Hooshmand et al., 2013).

Timber Structures Application

Although most SMA research has primarily focused on steel structures, recent studies reveal potential benefits for wood systems. Huang and Chang (2017) used Cu-Al-Mn SMA tubes and dowels in double-shear timber joints (Figure 2.21a), resulting in minor self-centering and steady hysteresis. Similarly, embedding comparable dowels in densified-veneer wood, DVW, (Figure 2.21b) reduced local crushing while increasing both strength and recentring capacity (Huang et al., 2019). Comparative testing showed that NiTi dowels outperform Cu-based tubes for recentring; however, the tests were confined to unidirectional cycling (Schwarze et al., 2022). In addition, traditional Chinese mortise-and-tenon frames modified with NiTi wires demonstrated increased rigidity, ultimate higher capacity, and significantly decreased residual drift during earthquake simulations (Xie et al., 2019, 2020).



a) connections with SMA tubes/bars
Retrieved from Huang and Chang (2017)

b) connections with SMA dowels and DVW
Retrieved from Huang et al. (2019)

Figure 2.21. Test Setup for Double Shear Connections with SMA Dowels

To date, the potential of implementing SMA dowel-type connections in glulam BTF system has been investigated by Cl  roux et al. (2024). The authors tested 24 dowel joints with either A307 steel or NiTi SMA bolts under ASTM E2126 protocols. The SMA connections retained only half the initial stiffness of steel joints while increasing yield and ultimate strength by 1.3-2.0 times. Their narrow flag-shaped loops achieved ductility ratios of $\mu \approx 4.6$ (four bolts) and $\mu \approx 6.3$ (single bolt), exhibiting little residual drift, meeting limited ductility but not moderate-ductility requirements. Post-test inspections indicated that only the central NiTi hinge remained detwinned, while the other hinge sections self-centred fully, demonstrating the unique recovery process of SMA dowel-type connections (Cl  roux et al., 2024). The methodology and findings from this study will be further reviewed in Section 3.1, given their importance for the work presented in this research project.

2.5 Modelling in OpenSees

While experimental investigations have shown that SMA connections in BTFs have the potential to minimize permanent deformations after earthquake loading, numerical analysis provides a more cost-efficient way to investigate a larger variety of parameters and stress conditions. Open-source software, such as OpenSees (Open System for Earthquake Engineering Simulation), has been used by researchers to simulate the seismic reaction of structures, including those made of novel materials like SMA (Dong et al., 2019). OpenSees is an object-oriented software framework for developing seismic response simulation applications for structural and geotechnical systems (McKenna, 2011). This software dates its origin to McKenna's (1997) doctoral thesis, and then its framework was made public in 2000 with the creation of the opensees.berkeley.edu website (McKenna et al., 2000). The Command Language Manual (Mazzoni et al.,

2006) provides detailed user guidance on element and material libraries. Recently, the publication by McKenna et al. (2010) is the most commonly referenced peer-reviewed source in academic publications.

In earthquake engineering applications, OpenSees provides a framework for engineers and researchers to develop finite element models to experiment with different modelling tools, analytical methods, and solution algorithms (McKenna, 2011). This software is based on C++, but it incorporates historical Fortran code and interfaces with numerical libraries written in C or Fortran, increasing its computational efficiency and adaptability (McKenna et al., 2010). It can be used in practical applications; however, it was primarily designed so that researchers may incorporate reliability approaches into the applications (McKenna, 2011).

The present study employs OpenSees v3.7.1, downloaded from their public website, and reference to the software OpenSees will be used without repeated citation for simplification. However, all numerical modelling and analysis were carried out through OpenSeesPy, the Python interpreter layer for OpenSees developed by Zhu et al. (2018). Their work presented a new interface for the OpenSees framework as a *DL_Interpreter* class to connect not only the extensive Python scientific ecosystem (NumPy, SciPy, etc.) but also allow for possibility to accommodate other scripting languages such as Ruby, Julia, or R. Python allows common structures like loops, conditionals, and user-defined functions, and it avoids the string-based syntax inherited by OpenSees from Tcl, a limitation that makes mathematical operations awkward and increases the learning barrier for new users (Zhu et al., 2018).

OpenSees excels at modelling frame elements, connectors, and nonlinear springs, and its material models library allows for specific hysteresis behaviour (McKenna, 2011). However, OpenSees is not intended as a commercial application and does not have some of the integrated elements and materials found in the most powerful commercial software (McKenna, 2011). Despite these limitations, OpenSees helps in the development and sharing of new material and element routines, solvers, integration schemes, and nonlinear solution algorithms (McKenna, 2011). OpenSees can perform static pushover, static reversed cyclic, and dynamic time-series analyses (Lukic et al., 2018).

The numerical modelling of nonlinear connections and elastic wood elements can be easily integrated in OpenSees, as it allows for detailed simulation of experimental results under seismic loads (Dong et al., 2021). For BTJs, the brace connections in their principal direction (parallel-to-grain axis) can be represented by a uniaxial nonlinear spring to allow for the capture of slip, pinching, and self-centering, while the perpendicular-to-grain axis could be maintained elastic because of the minimal transverse stresses during seismic loading, effectively representing a uniaxial connection.

2.5.1 Uniaxial Spring Models

OpenSees presents numerous uniaxial material models (referred to as spring models) that theoretically characterize the envelope and hysteresis of timber joints using a key parameter (Dong et al., 2019). The most used option is SAWS (Seismic Analysis of Wood-frame Structures), which was developed by Folz and Filiatrault (2001a, 2001b). This model, categorized as the CUREE (California Universities for Research in Earthquake Engineering) 10 parameter model, has been adopted widely in timber structures due to its ability to replicate the pinching effect of hysteretic curves and the degradation of strength and stiffness with fewer parameters (Liu & Yang, 2022). SAWS has also been validated against experimental testing for dowel-type joints in numerous studies (Cao et al., 2019a; Folz & Filiatrault, 2004b; Li et al., 2018a; Pozza et al., 2017; Y.-L. Shen et al., 2013). A second option is Pinching4, a four-point pinching model developed for reinforced-concrete beam-column joints (Lowes et al., 2004) but later used to capture the cyclic response of dowel-type timber connections (Cao et al., 2019b; Dong et al., 2021; Li et al., 2018b; Pozza et al., 2017; Y. L. Shen et al., 2013). These models provide researchers with a variety of options for modelling timber-joint behaviour.

However, these existing models sometimes cannot capture the full spectrum of nonlinear behaviour exhibited by dowel-type joints under cyclic loading. To overcome these gaps, Dong et al. (2021) introduced a versatile uniaxial model for dowel-type connections, the DowelType spring model, that has since been validated across multiple wood-connection studies (Carrero et al., 2024; Chacón & Guindos, 2023; Contiguglia et al., 2024; Dong et al., 2024; X. Wang et al., 2023). The present work adopts both SAWS and DowelType to represent brace fasteners, as these were specifically developed for dowel-type timber connections. Detailed information for both material models is presented below.

SAWS Material Model

Folz and Filiatrault (2001a, 2001b) developed the SAWS model as part of their studies to predict the dynamic properties, quasistatic pushover, and seismic response of wood-frame structures. The first journal paper by Folz and Filiatrault (2004a) details the SAWS model formulation, and it is part of two companion papers. In this first part, the model formulation is presented as a model to capture the nonlinear lateral load-resisting response of shear wall elements with nailed connections in light-frame wood structures. To reduce computational complexity, the three-dimensional structure is condensed into a two-dimensional scheme linking floor and roof diaphragms, or the foundation, through zero-length shear wall spring elements (Folz & Filiatrault, 2004a). Each shear wall is represented by a single degree of freedom (SDOF) element, calibrated to reflect its hysteretic behaviour. These SDOF shear spring elements can exhibit

pinching as well as strength- and stiffness-degradation, and represent a model with symmetrical behaviour as shown in Figure 2.22 with their parameters (PM) shown in Table 2.3. Monotonic and cyclic envelopes are expressed by a combination of an exponential function and a descending linear line, originally proposed by Foschi (1977) in equations (2.1), (2.2), and (2.3).

$$F = \text{sgn}(\delta)(F_0 + R_1 S_0 |\delta|)[1 - \exp(-S_0 |\delta|/F_0)], \quad |\delta| \leq |D_U| \quad (2.1)$$

$$F = \text{sgn}(\delta) \cdot F_{peak} + R_2 S_0 [\delta - \text{sgn}(\delta) D_U], \quad |D_U| < |\delta| \leq |D_F| \quad (2.2)$$

$$F = 0, \quad |\delta| > |D_U| \quad (2.3)$$

where, F_0 : intercept strength for the asymptotic line to the envelope curve $F_0 > F_I > 0$
 D_U : Spring element displacement at ultimate load. ($D_U > 0$)
 D_F : Failure displacement
 S_0 : Initial stiffness of the hysteretic curve ($S_0 > 0$)
 R_1 : Stiffness ratio of the asymptotic line to the envelope curve ($0 < R_1 < 1$)
 R_2 : Stiffness ratio of the descending segment of the envelope curve ($R_2 < 0$)
 δ : Strain.

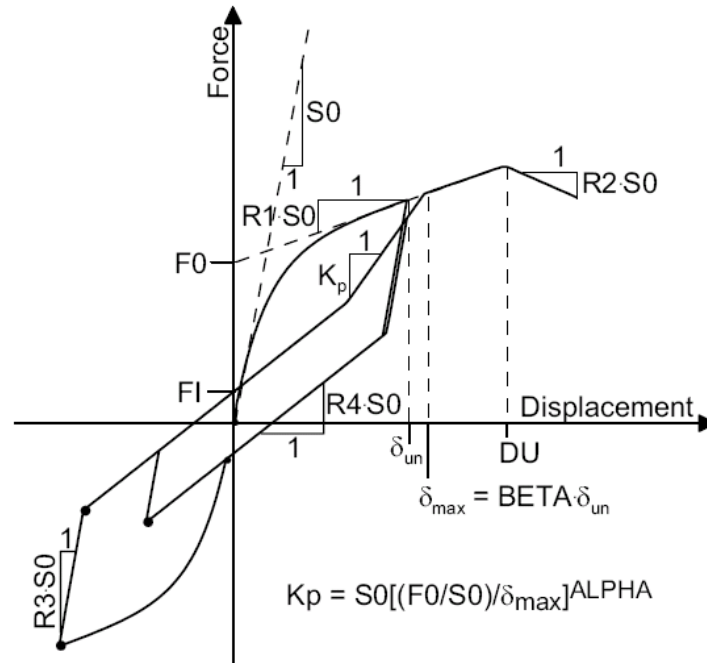


Figure 2.22. SAWS Material Model
Retrieved from Folz and Filiatrault (2001b)

*Table 2.3 SAWS Material Model Parameters
Retrieved from Folz and Filiatrault (2001b)*

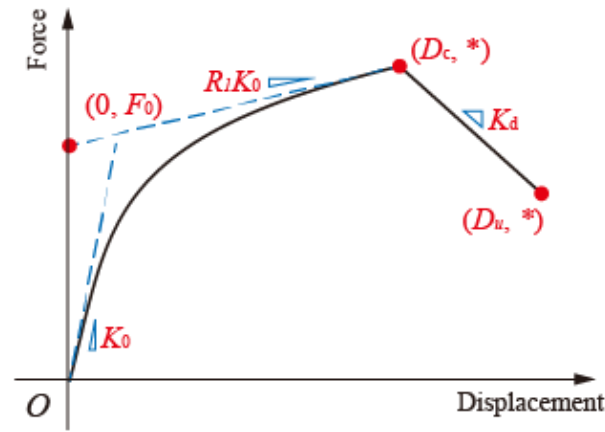
PM	Description
F_0	Intercept strength of the shear wall spring element for the asymptotic line to the envelope curve $F_0 > F_1 > 0$
F_1	Intercept strength of the spring element for the pinching branch of the hysteretic curve. ($F_1 > 0$)
D_U	Spring element displacement at ultimate load. ($D_U > 0$)
S_0	Initial stiffness of the shear wall spring element ($S_0 > 0$)
R_1	Stiffness ratio of the asymptotic line to the spring element envelope curve ($0 < R_1 < 1$)
R_2	Stiffness ratio of the descending branch of the spring element envelope curve ($R_2 < 0$)
R_3	Stiffness ratio of the pinching branch for the spring element
R_4	Stiffness ratio of the pinching branch for the spring element ($R_4 > 0$)
alpha	Stiffness degradation parameter for the shear wall spring element. ($\alpha > 0$)
beta	Stiffness degradation parameter for the spring element ($\beta > 0$)

In Folz and Filiatrault (2004b), the SAWS model was applied and verified by comparing its numerical predictions to experimental results from shake table testing of a full-scale two-storey light wood-frame house. The test structure, which represented a simpler two-storey single-family house, consisted of 10 different phases. However, only phases 9 and 10 of the shake table test structures were utilized to validate the SAWS model. Numerical simulations accurately captured the dynamic characteristics, pushover strength, and seismic demands, with peak relative displacement predictions averaging within 10% of the experimental values. Some discrepancies were observed in the SAWS definition of stiff diaphragms, which had a significant impact on phase 9 results but had a minimum impact on phase 10. From this study, Folz and Filiatrault (2004b) conclude that the SAWS model is a useful and simple tool for structural analysis and research to reliably estimate the modal behaviour, pushover response, and seismic performance of wood-frame structures.

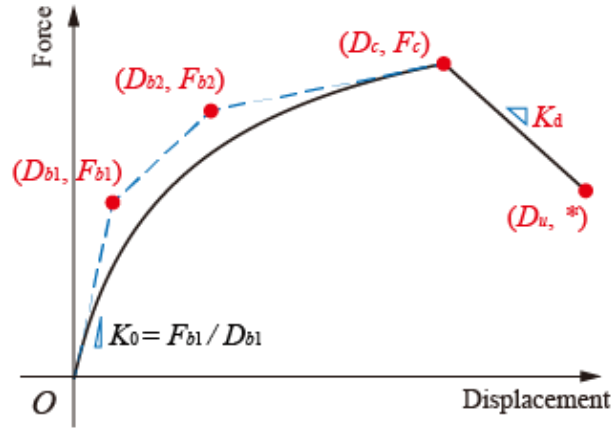
DowelType Material Model

Dong et al. (2021) developed the DowelType mode, a novel phenomenological uniaxial hysteretic model for dowel-type timber connections that separates the envelope law from the hysteretic rule, allowing each to be set individually. DowelType provides the accuracy needed to predict the dowel-type connection load-deformation behaviour, which is crucial for seismic assessment of timber structures, without sacrificing computing performance (Dong et al., 2021). In this material model, the envelope curve, which describes the overall force-displacement relationship, includes three options: a) exponential, b) Bézier, and c) piece-wise linear, as shown in Figure 2.23. The exponential envelope is based on Foschi's (1977) work, equations (2.1),(2.2), and (2.3), for the ascending curve and on a linear function for the descending curve (similar to

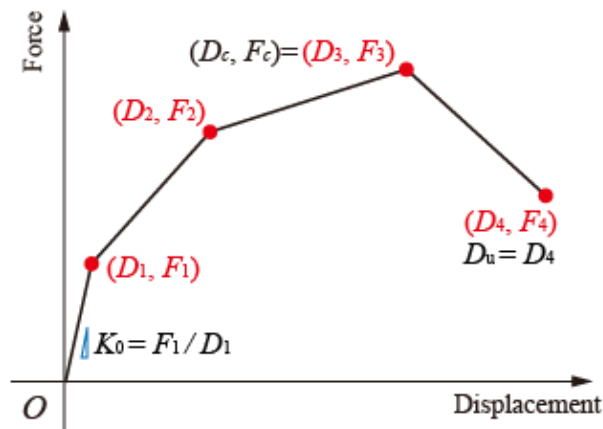
the SAWS model). Bézier curves allow numerical continuity and shape control through parametric equations, whereas piecewise linear interpolation points are linearly connected (Dong et al., 2021).



a) Exponential Model



b) Bézier Curve Model



c) Piece-wise Linear Model

Figure 2.23. DowelType Envelope Models
Retrieved from Dong et al. (2021)

In addition, the hysteretic law governing the cyclic behaviour, builds upon the SAWS model (Figure 2.24a) but incorporates Bézier curves for smoother transitions between slopes (Figure 2.24b). The pinching line, defined by the force intercept, F_l , and the pinching stiffness, K_p , varies with unloading point coordinates. This formulation, together with stiffness degradation models, allows the DowelType model to describe the complex hysteretic behaviours observed in timber joints.

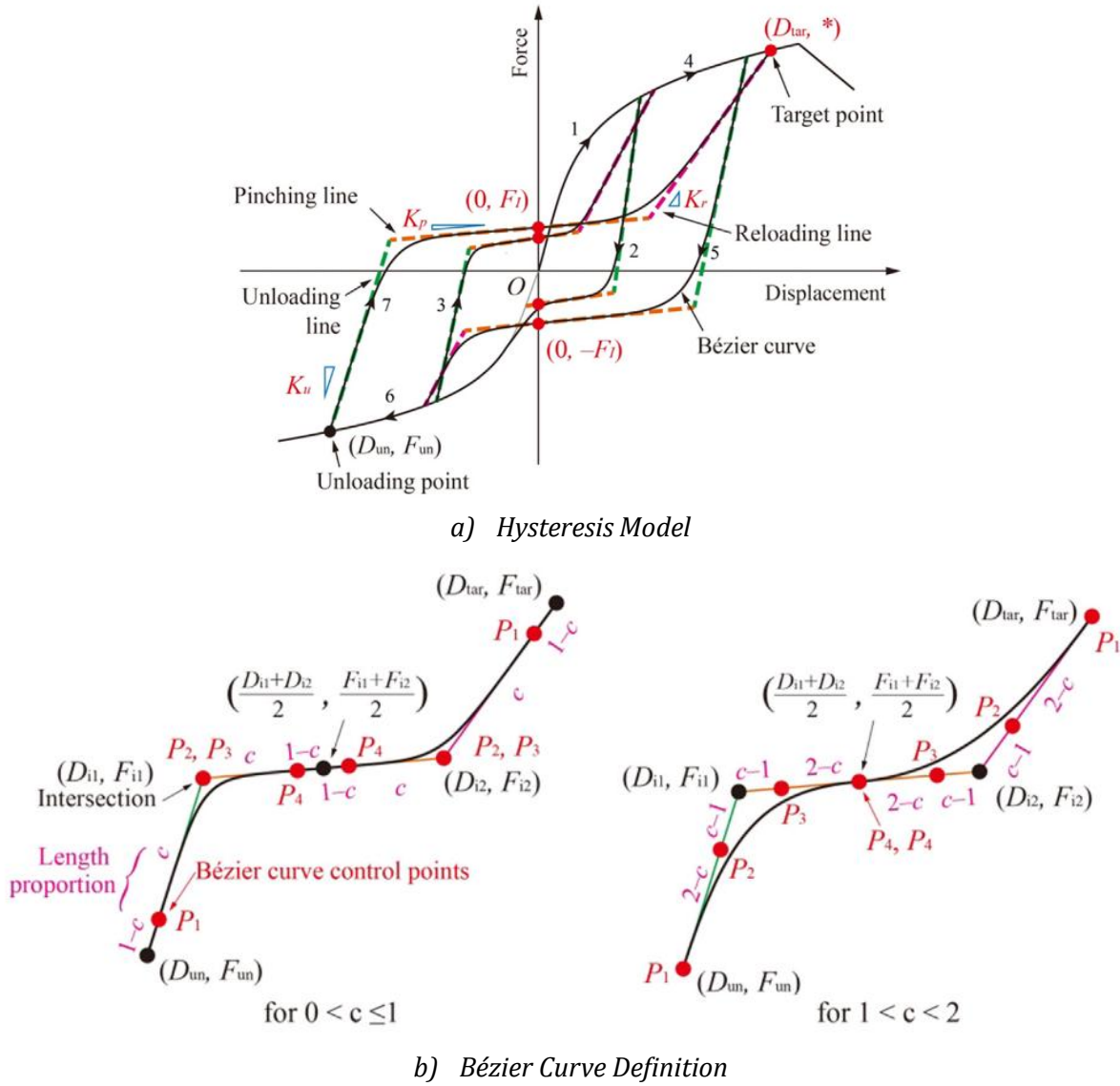


Figure 2.24. DowelType Hysteretic Law Model
Retrieved from Dong et al. (2021)

The parameters that define the DowelType envelope and hysteretic curves are specified in Table 2.4.

Table 2.4. DowelType Model Parameters
Retrieved from Dong et al. (2021)

PM	Description
Universal	
F_i	Pinching line force intercept
K_p	Stiffness of the pinching line
R_u	Unloading stiffness to initial stiffness ratio
c	Hysteresis curvature factor. Curvature increases when c is larger ($0 \leq c < 2$)
β	An amplification factor of the maximum displacement to account for maximum displacement based on reloading stiffness and strength degradation ($\beta \geq 1$)
γ	A factor related to the energy-based damage index to account for energy based reloading stiffness and strength degradation ($\gamma \geq 1$)
η	A factor related to the extent that the force intercept varies with the maximum displacement ($\eta \geq 1$)
D_y	Apparent yielding displacement
α_p	Stiffness degradation factor for pinching line
α_u	Stiffness degradation factor for unloading line
α_r	Stiffness degradation factor for reloading line
Exponential	
K_0	Initial stiffness
R_1	Stiffness for the asymptotes of the envelope to initial stiffness ratio
F_0	Envelope asymptotes force intercept
D_c	Cap displacement of the envelope
K_d	Absolute descending stiffness
D_U	Ultimate displacement
Bézier	
D_{b1}	The displacement of the first controlling point of the Bezier curve
F_{b1}	The force of the first controlling point of the Bezier curve
D_{b2}	The displacement of the second controlling point of the Bezier curve
F_{b2}	The force of the second controlling point of the Bezier curve
D_c	Cap displacement of the envelope
F_c	Cap force of the envelope
K_d	Absolute descending stiffness
D_U	Ultimate displacement
Piece-wise	
D_1	Displacement of the 1st piecewise linear envelope curve interpolation point
F_1	Force of the 1st piecewise linear envelope curve interpolation point
D_2	Displacement of the 2nd piecewise linear envelope curve interpolation point
F_2	Force of the 2nd piecewise linear envelope curve interpolation point
D_3	Displacement of the 3rd piecewise linear envelope curve interpolation point
F_3	Force of the 3rd piecewise linear envelope curve interpolation point

2.5.2 Frame-Level Numerical Models with SMA Connections

Dong et al. (2019) performed numerical analyses on steel frames with self-centering energy dissipation braces that included SMA cables. They began by investigating the mechanical properties of SMA cables to develop the appropriate numerical models, as shown in Figure 2.25. They then presented a SMA self-centering energy dissipation brace (SMA-SCEB) device and measured its hysteretic behaviour using quasi-static cycle loading experiments. The device was subsequently installed in a steel-framed structure, and nonlinear dynamic analyses were used to evaluate its seismic performance. The results showed that SMA-SCEBs greatly improved the structure's residual drift ratio when compared to braces with basalt fiber-reinforced polymer cables (BFRP-SCEBs). The steel frame with SMA-SCEBs also achieved a lower peak acceleration than the one with BFRP-SCEBs (Dong et al., 2019).

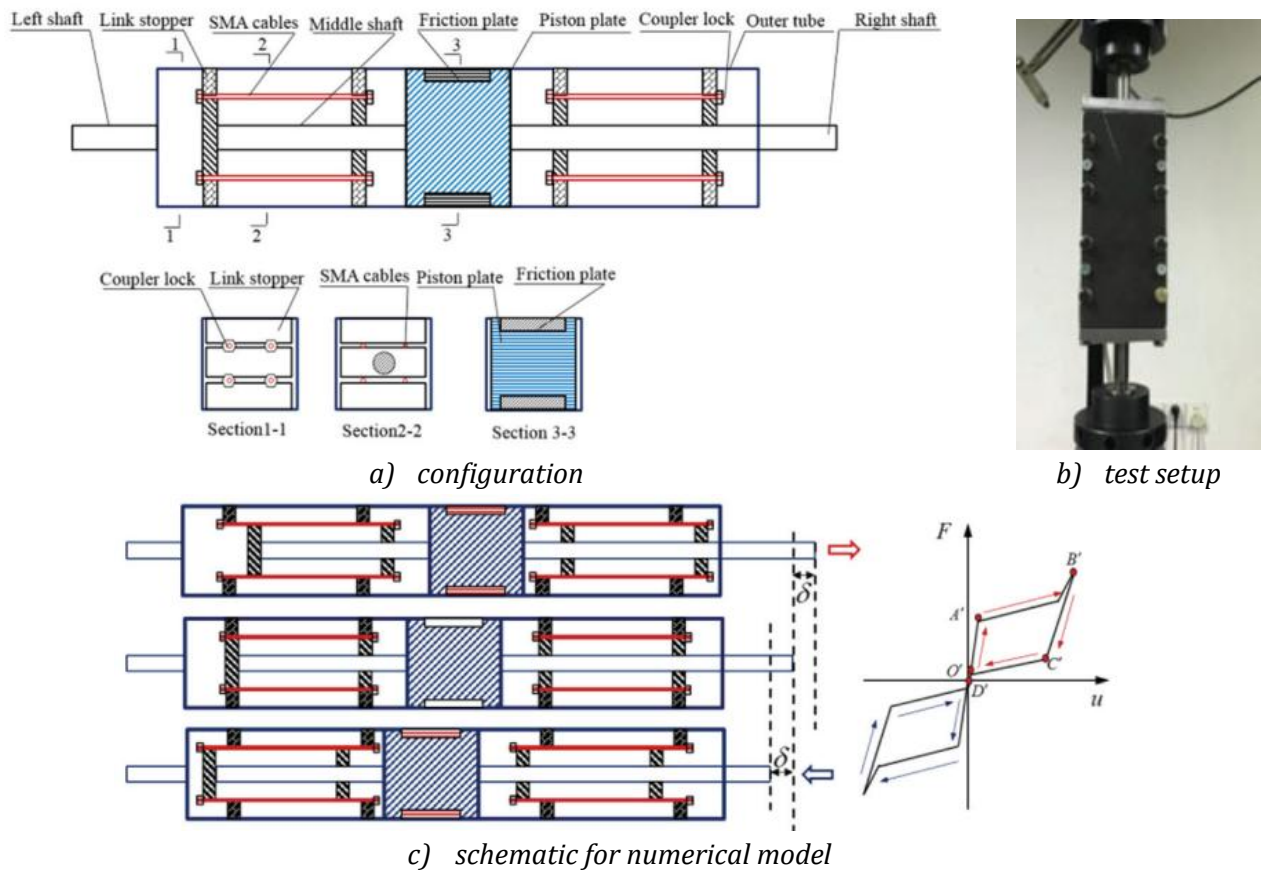


Figure 2.25. Modified SCEB based on SMA cables
Retrieved from Dong et al. (2019)

Mahmoudi et al. (2018) used OpenSees to model three, five, and seven-storey steel X-knee-braced frames (X-KBFs), which included SMA bars between beam-column joints and knee members as shown in Figure 2.26. They evaluated structural qualities like stiffness, ductility, strength, and recentring capability under

seismic loading. Pushover tests indicated the effect of SMA bars on plastic hinge formation. The study discovered that SMA bars did not significantly modify stiffness, ductility, or strength, but they significantly enhanced self-centering, lowering permanent roof displacements by up to 90% (Mahmoudi et al., 2018). The models used conventional industry parameters for NiTi material parameters and showed that plastic strains in all SMA bars remained below the collapse-prevention threshold (Mahmoudi et al., 2018).

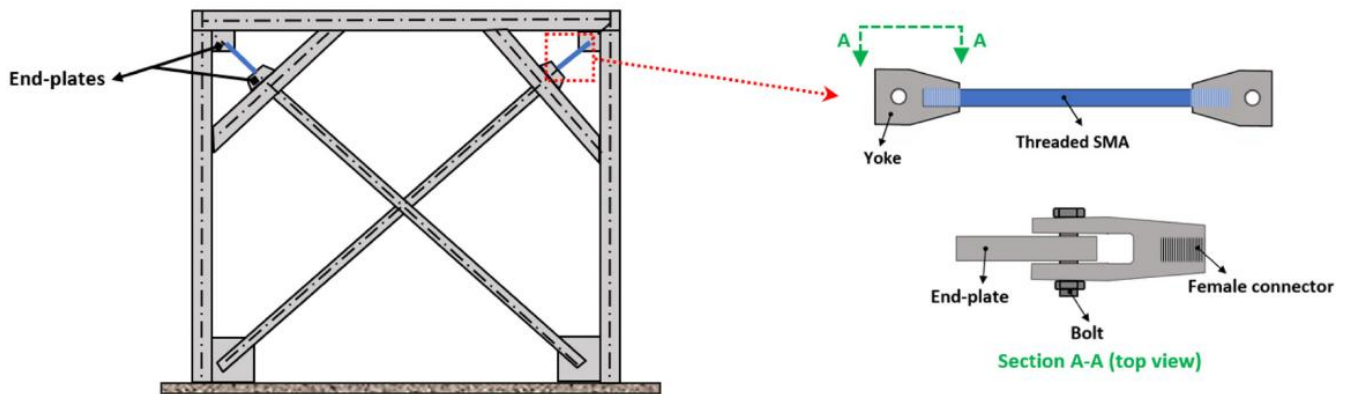


*Figure 2.26. SMA bars at knee brace connections
Retrieved from Mahmoudi et al. (2018)*

Babaei and Zarfam (2019) used OpenSees to optimize SMA braces inside Eccentrically Braced Frames (EBF) in steel structures of three- and six-storey frames. They used a time-based approach to easily compare many frames and examine the effect of SMA length on structural behaviour (see Figure 2.27). The study performed the analysis in OpenSees and compared the results in MATLAB to assess the maximum interstorey and residual roof drifts of the various frames, resulting in drifts of less than 0.8 mm in some circumstances while retaining consistent stiffness. In this study, dual-braced frame configurations met safety criteria, with a 25% reduction in interstorey drift compared to traditional steel frames (Babaei & Zarfam, 2019). The probabilistic and deterministic results, combined with the performance level, revealed that even a tiny quantity of SMA can increase seismic response (Babaei & Zarfam, 2019).

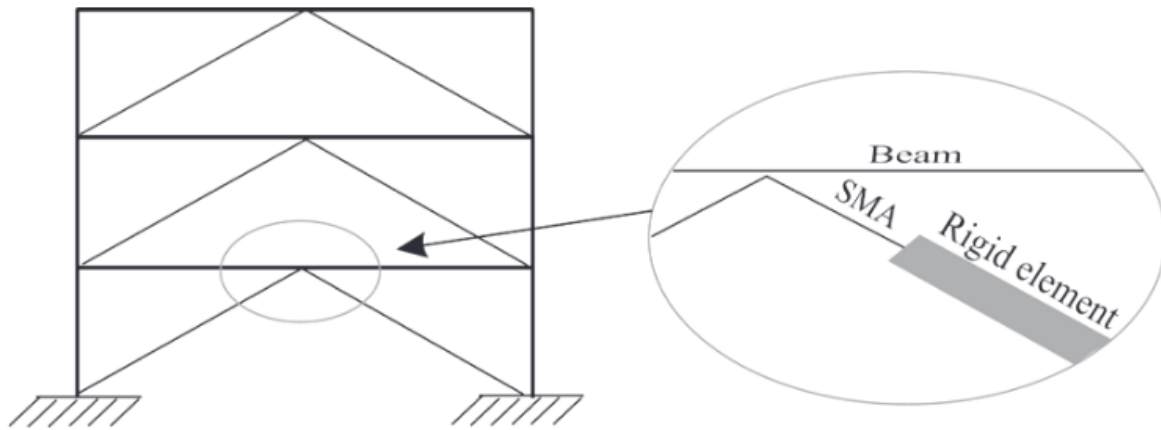


*Figure 2.27. SMA brace connection
Retrieved from Babaei and Zarfam (2019)*

Mirzai and Attarnejad (2018) proposed a Recentring Damping Device (RDD) with SMA bars on EBFs steel structures and performed numerical tests on a nine-storey steel frame. Figure 2.28 shows the RDD with SMA bars modelled in this study. The RDD device's non-linear time history analysis done in OpenSees with ten ground motions, revealed a 57% reduction in the highest interstorey drift ratio when the RDD system was utilized (Mirzai & Attarnejad, 2018). Furthermore, the RDD system reduced residual drift by up to 86.99% when compared to conventional EBFs, resulting in little residual displacement (Mirzai & Attarnejad, 2018). Pushover analysis revealed that the RDD system has a higher capacity than the conventional system. This study demonstrates the potential of SMA-based dampers to improve the seismic resilience of structures by minimizing both immediate and permanent deformations.

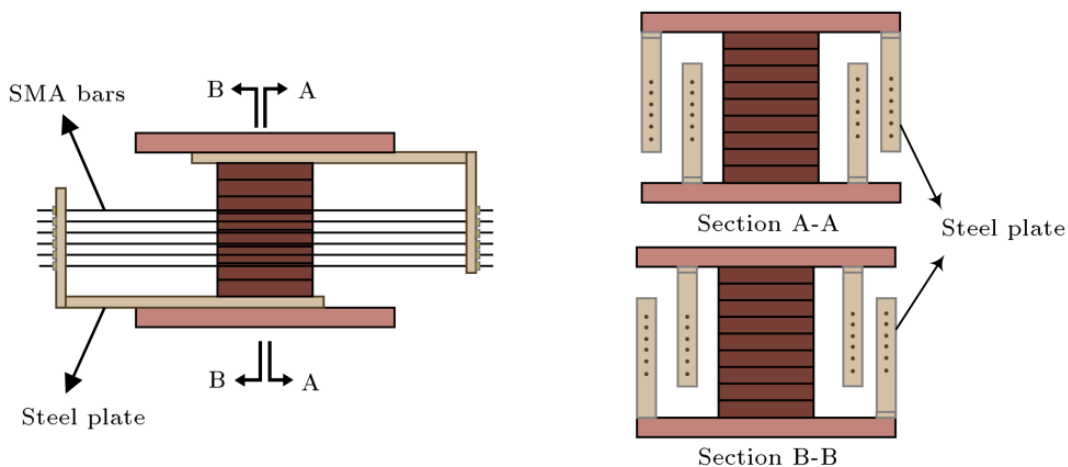


*Figure 2.28. Re-centering Damping Device (RDD) with SMA bars
Retrieved from Mirzai and Attarnejad (2018)*

2.6 Summary

The growing adoption of mass timber buildings has increased the need for reliable LLRS, especially BTFs in moderate seismic zones. A significant research interest has focused on optimizing the performance of BTF connections to provide adequate ductility to dissipate earthquake energy, prevent brittle timber failure, and achieve seismic resilient structures. Although more sophisticated connectors exist, dowel-type fasteners remain the industry standard because of their simple detailing and code compliance, highlighting the importance of advanced modelling and design guidelines.

Current standards, such as the CSA O86, offer no guidance on ductility criteria for these dowel-type connections, contributing to uncertainty in meeting the seismic performance requirements and structural resiliency post-seismic events. Recently, cyclical testing of BTF connections proved the self-centering potential of SMA bolts compared to traditional steel fasteners. However, evaluating the behaviour of SMA bolts at a system-level for BTF structures to confirm the self-centering potential shown at a connection-level is still needed. Advances in numerical modelling now make such evaluations feasible without extensive experimental work, giving researchers a practical means to predict the seismic response of BTFs.

CHAPTER 3. PROOF OF CONCEPT

3.1 Overview

Chapter 3 presents a numerical proof-of-concept framework for BTFs, demonstrating how connection-level models calibrated using experimental results can replicate their cyclic behaviour of SMA and steel dowel-type connections. Connection parameters extracted from the four-bolt tests reported by Cl  roux et al. (2024) were used to calibrate the connection-level models with SAWS and DowelType uniaxial spring materials available in the OpenSees library. In this study, the OpenSeesPy interpreter was used for ease in the development of the code. These spring models were validated at the connection-level, then used in brace-level models to implement the concept of stronger and weaker connection behaviour. Building upon these models, frame-level pushover monotonic analysis confirmed that the presented modelling approach estimated ultimate displacements within $\pm 5\%$ of results found using theoretical methods.

Finally, the frame-level models were developed into multi-storey models of three and four storeys that were subject to nonlinear time history analyses under fifteen scaled ground motions. The results showed that the SMA connections reduced the residual drift by approximately 90% while maintaining comparable strength. This confirmed the self-centering ability of SMA connections and proved to be a suitable numerical modelling method to develop full-scale system-level BTF models. In sum, Chapter 3 focuses on a numerical modelling framework that can capture the self-centering potential observed at a connection-level to develop system-level models. This provides a robust methodology to expand the modelling techniques and develop prototype system-level frames for a comprehensive seismic performance evaluation.

3.2 Reference Experiment

3.2.1 Overview

The experimental study conducted by Cl  roux et al. (2024) on dowel-type bolted connections aimed to investigate the potential of Nickel-Titanium (NiTi) SMA dowels in dowel-type connections. These connections were designed with slotted-in steel plates in glulam braces for BTF systems with the aim of presenting an alternative to typical steel bolted connections. The research examined the connection-level behaviour of dowel-type connections through 24 experimental tests, comprising 12 single-dowel (6 steel, 6 SMA) and 12 four-bolt (6 steel, 6 SMA) configurations.

The research program of Cl  roux et al. (2024) for the 12 four-dowel connections were designed to represent reduced-scale connections used in multi-storey BTF systems, making them particularly relevant to this study. Half of the connections were loaded monotonically (3 steel, 3 SMA) while the remainder were tested cyclically (3 steel, 3 SMA). According to Cl  roux et al. (2024), the SMA connections showed significant self-centering potential under cyclic loading, with minimal permanent deformation after unloading. In contrast, the steel connections displayed noticeable permanent deformations, indicating their inability to self-center under cyclic loading.

3.2.2 Experimental Program

Cl  roux et al. (2024) conducted monotonic and cyclic loading tests on half-brace members utilizing an MTS 322 hydraulic frame, as shown in Figure 3.1. The timber half-brace is a 127 x 140 x 500 mm 20f-EX glulam member, anchored to the frame table with a connection comprised of two steel angles and 3/4-inch A307 steel bolts. This connection was oversized to ensure displacement of the connection of interest only. At the top of the frame, the 300W steel plate was held by the actuator's hydraulic grip. An internal load cell within the actuator recorded the applied loads during testing. The displacements were recorded using two linear variable differential transformer (LVDT) sensors, mounted at the top of the glulam specimen, with their spring cores secured by aluminum angles fixed to the steel plate. The displacement measurements were taken as the average of the readings from both LVDT sensors.

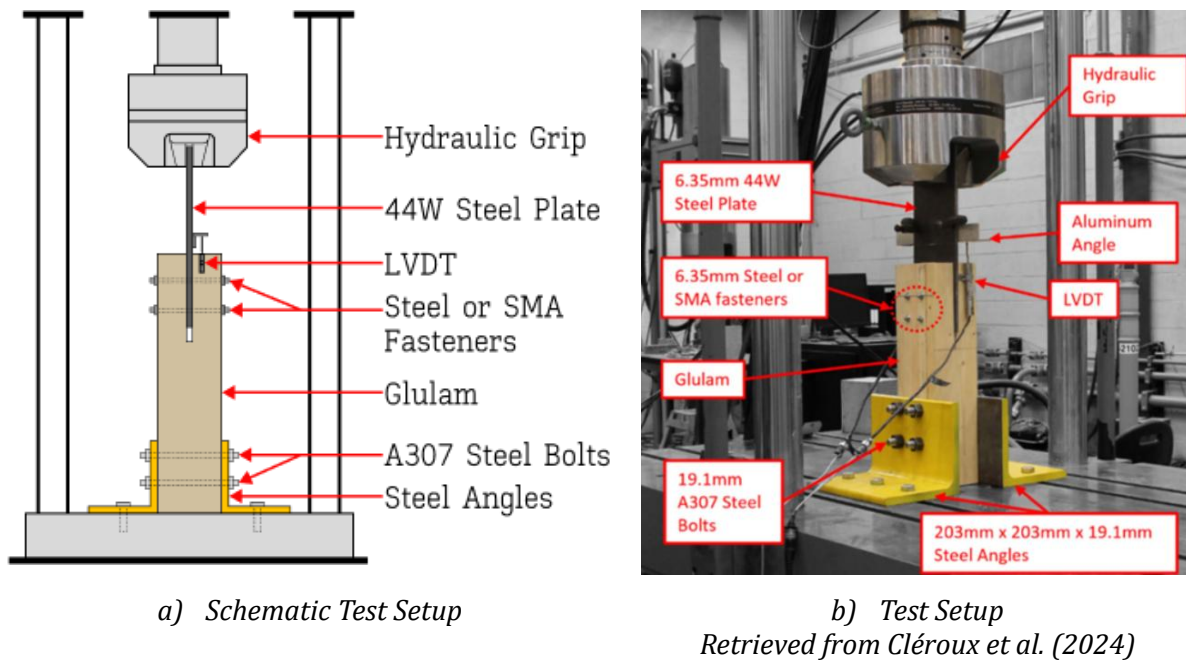


Figure 3.1. Experimental Test Setup

The connection specimens are shown in Figure 3.2 and represent a reduced-scale wood-steel-wood glulam connection featuring an internal 6.35 mm thick 300W steel plate and four 6.35 mm diameter fasteners representative of a connection used in multi-story BTF buildings. The fasteners consist of steel or shape memory alloy (SMA) bolts, and the design aims to achieve ductile failure modes in accordance with the CSA O86 standard (2019). The glulam members are slotted to accommodate the internal steel plate, and a 20 mm gap is left at the end of the steel plate to prevent the plate from bearing on the wood when the connection is in compression. The four-bolt configuration uses four evenly spaced fasteners to secure the steel plate.

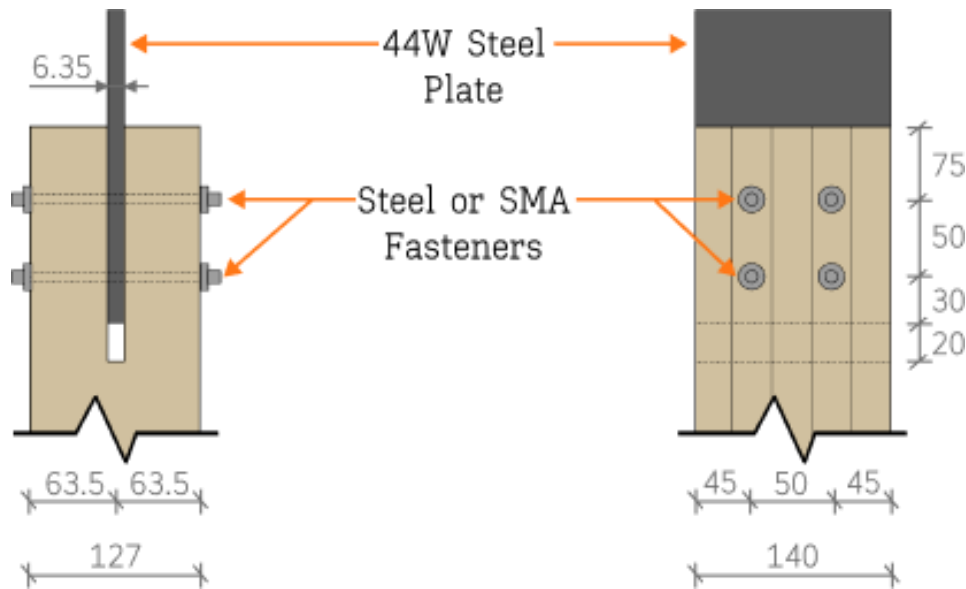


*Figure 3.2. Four-Bolt Connection
Reproduced from Cl  roux et al. (2024)*

The connections were tested under both monotonic and cyclic loading protocols following ASTM Standard E2126 (2019). In the monotonic tests, both materials were loaded at a rate of 4 mm/min, and load and displacement data were recorded at 10 samples per second until a displacement of 45 mm was reached or failure occurred. Failure in the monotonic tests was defined as a 20% reduction in load-carrying capacity or the appearance of secondary failure modes.

For the cyclic tests, the protocol followed Test Method B in ASTM E2126 (2019) with a maximum displacement of 12 mm. The cyclic loading protocol consisted of two patterns. The first pattern applied displacement amplitudes ranging from 1.25% to 10% of the maximum displacement, while the second pattern increased the amplitudes from 20% to 120% of the maximum displacement. The tests subjected the connections to full reversed cyclic loading, producing data capturing elastic and inelastic behaviour.

*Table 3.1: Cyclic Loading Protocol of Reference Experiment
Reproduced from Cl  roux et al. (2024)*

Step	# Cycles	Amplitude	Displacement (mm)	Loading Rate (mm/min)
1	1	1.25%	0.15	2
2	1	2.5%	0.30	2
3	1	5.0%	0.60	2
4	1	7.5%	0.90	2
5	1	10.0%	1.20	2
6	3	20.0%	2.40	20
7	3	40.0%	4.80	20
8	3	60.0%	7.20	20
9	3	80.0%	9.60	80
10	3	100.0%	12.0	80
11	3	120.0%	14.4	80
12	3	140.0%	16.8	80
13	3	160.0%	19.2	80

3.2.3 Experimental Results

Presented herein are the results describing the global force-displacement responses as well as failure modes for the 6 steel and 6 SMA four-dowel connections tested under monotonic and cyclic loading. Figure 3.3 shows the force-displacement responses from the monotonic tests for both steel and SMA connections. The connections are observed to behave linearly up to the yield point, resulting in softening of the connection stiffness with a gradual increase in resistance up to failure.

Cl  roux et al. (2024) reported that monotonic force-displacement responses for steel and SMA bolted connections experienced significant deformations followed by wood failure in row shear parallel-to-grain. While a single specimen in each group was loaded to failure to identify ultimate behaviour, the remaining tests stopped earlier from investigating internal damage development. The steel connection observed a substantial decrease in resistance at 33 mm displacement, indicating the commencement of row shear failure. In contrast, the SMA connection failed at a lower displacement of about 17 mm (Cl  roux et al., 2024). As illustrated in Figure 3.3, SMA bolts exhibited higher average load resistance at 14 mm displacement, roughly 23% more than steel connections, despite having a 12% lower initial stiffness. The SMA connections also showed notable superelastic behaviour, including partial deformation recovery from hinge formation and localized crushing around bolt heads and nuts (Cl  roux et al., 2024). The formation of three plastic hinges in both steel and SMA connections indicated that Failure mode "g" described in the CSA 086 governed the overall behaviour of the connection (Cl  roux et al., 2024).

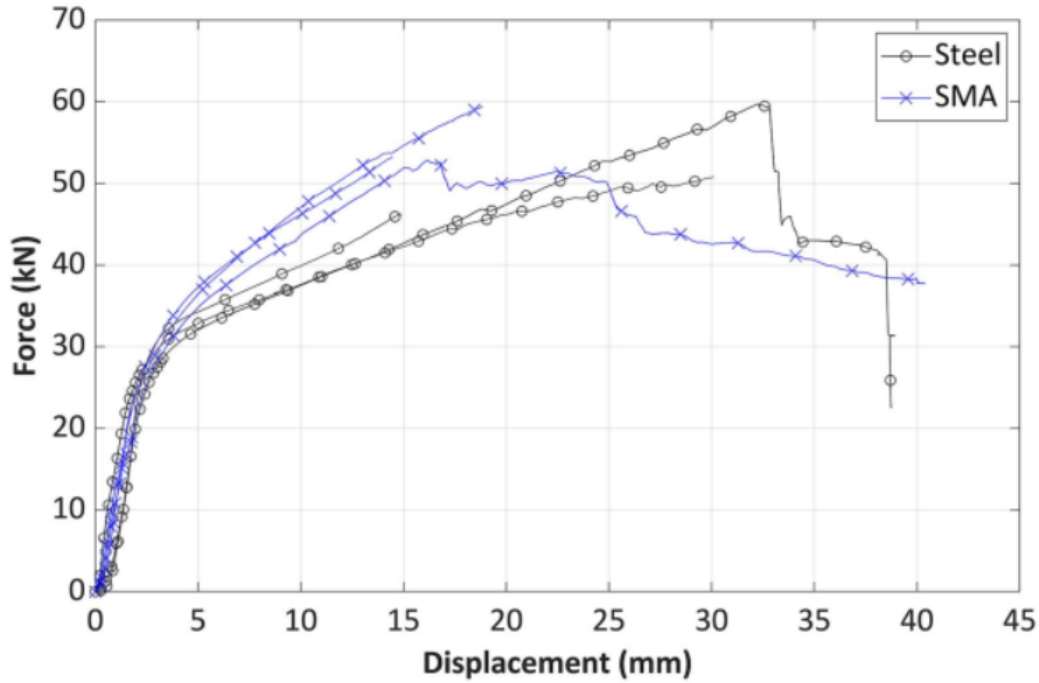
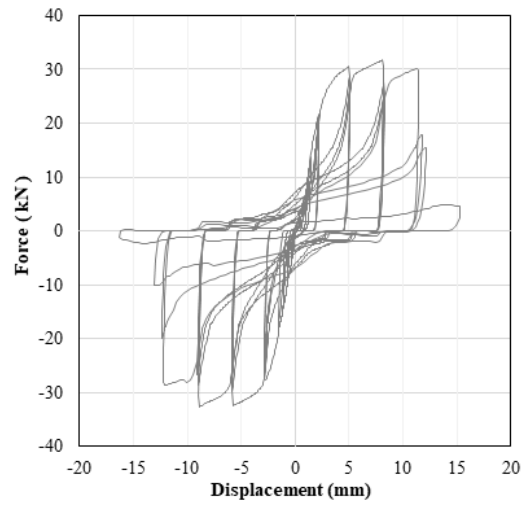
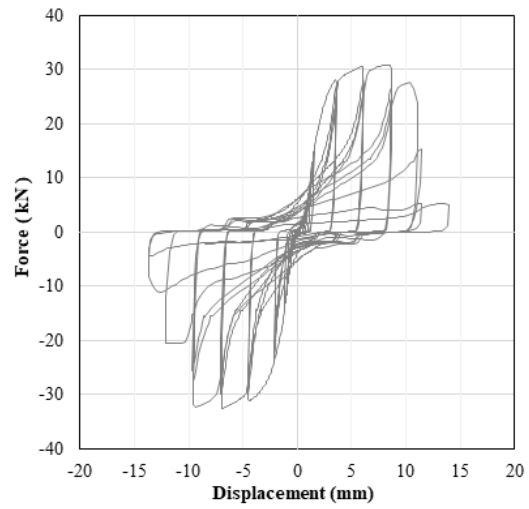


*Figure 3.3. Force-Displacement Curves for Monotonic Four-Bolt Connections
Retrieved from Cl  roux et al. (2024)*

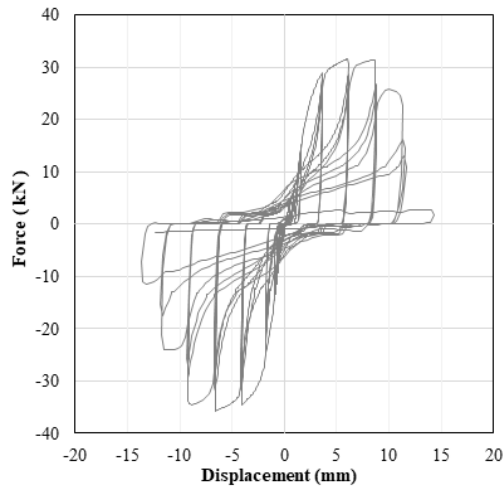
Regarding the force-displacement curves for the cyclic tests on the four-dowel connections, these are presented in Figure 3.4. The authors reported that the steel and SMA dowel connections demonstrated stable hysteretic behaviour under cyclic loading (Cl  roux et al., 2024). However, while the steel connections reached a strength plateau, the SMA connections continued to increase in strength until failure (Cl  roux et al., 2024). In addition, the steel connections exhibited wide hysteresis loops with unloading branches that deviated from the origin, indicating permanent deformation. In contrast, the SMA connections displayed pinched hysteresis loops with the unloading branches passing through zero displacements, indicating a self-centering response due to the superelastic properties of the SMA material (Cl  roux et al., 2024).



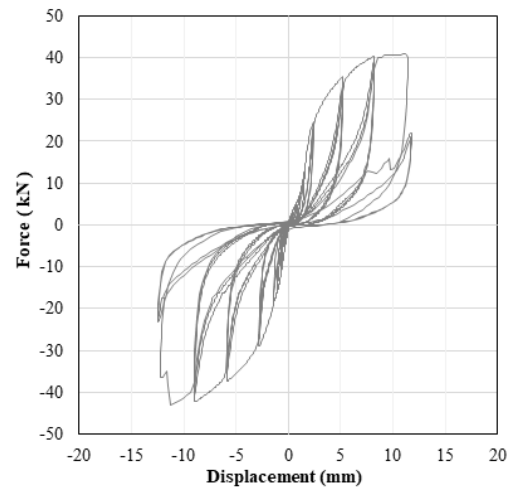
a) Steel 1



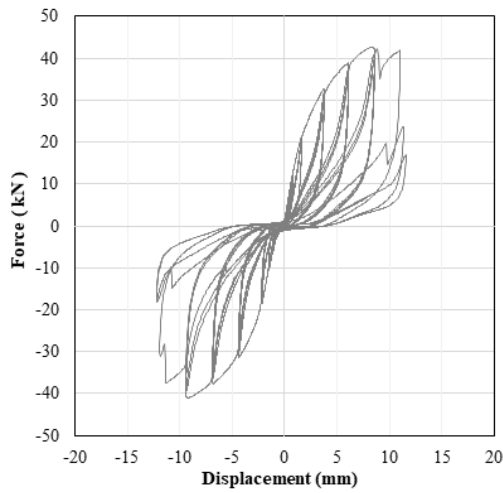
b) Steel 2



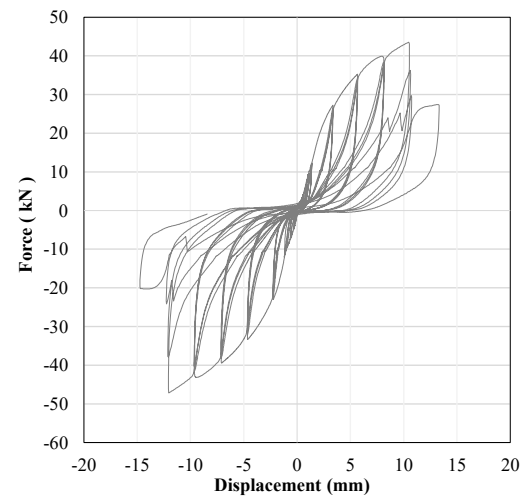
c) Steel 3



d) SMA 1



e) SMA 2



f) SMA 3

Figure 3.4. Hysteresis Curves Four-Bolt Connections
Reproduced from data provided by Cl  roux et al. (2024)

Both steel and SMA bolted connections demonstrated comparable deformation capabilities and failure displacements during cyclic loads (Cl  roux et al., 2024). The cyclic procedure used a maximum displacement (Δ_{max}) of 12 mm to capture early loading behaviour that may be disregarded if Δ_{max} is based on monotonic test results. Cl  roux et al. (2024) emphasize the importance of performing reverse cyclic testing, given that its results show the lowest capacity compared to the monotonic testing results. Even though SMA connections exhibited more strength, their failure was more abrupt than the progressive strength decline and breakage observed in steel bolts (Cl  roux et al., 2024). Similar to the monotonic tests, the development of three plastic hinges and their resulting wood crushing patterns indicated that failure in both connection types occurred in Mode "g" as per the CSA O86 (2019). In addition, Cl  roux et al. (2024) found more significant wood crushing in SMA connections. Although both bolt types broke at the central hinge, SMA bolts showed barely any residual deformation at the outer hinges due to superelastic recovery, whereas steel bolts exhibited substantial plastic deformation along their entire length (Cl  roux et al., 2024).

Table 3.2 summarizes the average results from the cyclic tests in both positive and negative loading directions. The yield strength of the SMA connections was at least 20% higher than that of the steel connections in the compressive direction, while their peak and ultimate loads were 30% greater. Despite this, the SMA connections exhibited lower stiffness compared to the steel connections. Cl  roux et al. (2024) also stated that the loads at yield, peak, and ultimate under cyclic testing were lower for both connection in comparison to monotonic tests. This reduction in strength is attributed to the fasteners breaking at lower displacements in cyclic tests, causing failure to occur earlier in the loading cycle (Cl  roux et al., 2024).

*Table 3.2. Connection Properties of Cyclic Connections
Reproduced from Cl  roux et al. (2024)*

Specimen	P_y^a (kN)	��_{Py}^b (mm)	P_{peak}^c (kN)	��_{Ppeak}^d (mm)	P_u^e (kN)	��_{Pu}^f (mm)	K^g (N/mm)	��^h
Steel								
1	19.7	1.6	31.1	7.8	24.9	11.1	13.5	7.0
2	13.3	1.0	29.8	5.7	23.9	10.6	13.0	10.4
3	13.5	1.1	30.9	5.7	24.7	10.6	11.7	9.4
Avg.	15.5	1.2	30.6	6.4	24.5	10.8	12.7	8.9
CoV	0.19	0.20	0.02	0.16	0.02	0.02	0.06	0.16
SMA								
1	23.6	1.8	39.9	7.7	31.9	11.0	13.8	6.2
2	25.3	2.4	41.8	8.6	33.4	11.2	12.5	4.6
3	23.2	2.8	42.7	10.6	34.1	12.1	9.0	4.2
Avg.	24.1	2.4	41.5	8.9	33.2	11.4	11.8	5.0
CoV	0.04	0.19	0.03	0.13	0.03	0.04	0.17	0.17
^a Yield load determined using the Y & K method				^e Ultimate load				
^b Yield displacement				^f Displacement at ultimate load				
^c Peak load				^g Stiffness of connection (slope from 10% to 40% of P _y)				
^d Displacement at peak load				^h Ductility defined as �� _{Pu} divided by �� _{Py}				

3.3 Brace-Level Model Development

3.3.1 Connection-Level Model

To accurately capture the behaviour of the connections, numerical connection-level models that replicate the experimental results from the work by Cl  roux et al. (2024) were developed. Plastic hinge (connection-level) models using zeroLength elements in OpenSees were defined to represent the non-linear behaviour of the connections. The connection-level models represent uniaxially loaded connection (Figure 3.5). The zeroLength element (11) represents the connections defined using two nodes (i.e., 1 and 2). However, node one is fixed while the second node is restrained in its rotation about the z-axis and its horizontal displacement (y-axis). The force-displacement behaviour in the x-direction between nodes one and two is defined through the spring material model assigned to the element.

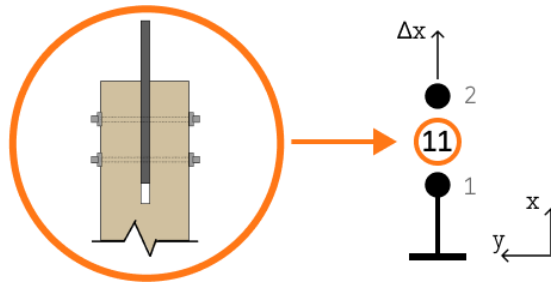


Figure 3.5. Schematic OpenSees - Plastic Hinge Model

The connection properties from the experimental testing of bolted steel and SMA connections provided in Table 3.2 were utilized for calibration purposes. Six four-bolt connections (i.e., 3 steel and 3 SMA) were calibrated to accurately replicate the nonlinear behaviour observed in the hysteresis curves. The steel connections were labelled as 4SC (4 bolts, steel, cyclic), while the SMA connections were designated as 4NC (4 bolts, SMA, cyclic), followed by a number from 1 to 3 assigned to each connection specimen based on the corresponding experimental test. The non-linear behaviour of the connections is defined using two uniaxial spring models available in the OpenSees library: (a) SAWS (Folz & Filiatrault, 2004a) and (b) DTE (Dong et al., 2021). The units of the models are kN, mm, and sec, and follow the cyclic loading protocol described in the experimental program in Table 3.1.

Two connection-level models with the average values found in the calibration of each material type were also calculated to obtain the average hysteresis curves (4NC_{avg} and 4SC_{avg}). Further information regarding the definition of each parameter has been provided in the literature review in Section 2.5.1. The parameters

used to calibrate the steel connections (4SC), and their average are provided in Table 3.3 and Table 3.4, respectively.

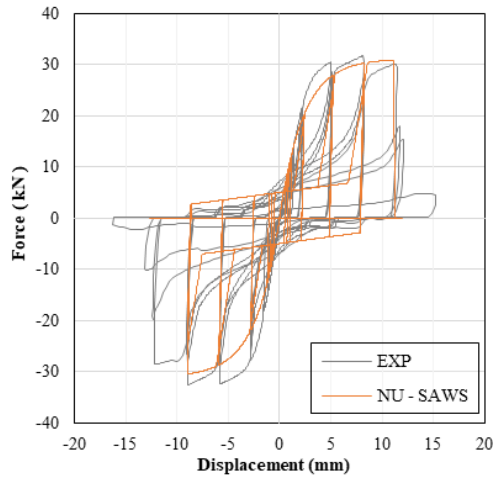
Table 3.3. Steel Connections - SAWS Model Parameters

Parameter		4SC1	4SC2	4SC3	4SC_{avg}
F0	kN	31.1	29.8	30.9	30.6
FI	kN	5.0	5.0	5.0	5.0
DU	mm	11.1	10.6	10.6	10.8
S0	kN/mm	13.5	13.0	11.7	12.7
R1		0.0	0.0	0.0	0.0
R2		-10.0	-5.0	-5.0	-6.3
R3		6.0	5.0	5.0	5.3
R4		0.02	0.03	0.03	0.026
alpha		0.06	0.06	0.06	0.06
beta		1.05	1.05	1.05	1.05

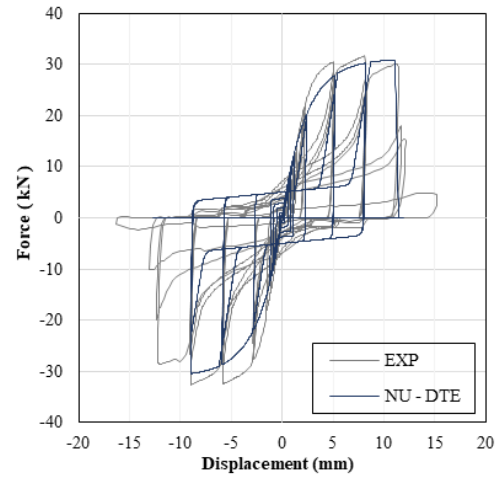
Table 3.4. Steel Connections - Dowel-Type Experimental Model Parameters

Parameter		4SC1	4SC2	4SC3	4SC_{avg}
Fi	kN	5.0	5.0	5.0	5.0
Kp		0.2	0.35	0.35	0.3
Ru		10.0	10.0	10.0	10.0
c		0.2	0.2	0.2	0.2
beta		1.02	1.02	1.02	1.02
gamma		1.05	1.05	1.05	1.05
eta		0.00075	0.00075	0.00075	0.00075
Dy	mm	1.6	1.0	1.1	1.2
alpha_p		0.02	0.02	0.02	0.02
alpha_u		0.05	0.05	0.05	0.05
alpha_r		0.05	0.05	0.05	0.05
K0	kN/mm	13.5	13.0	11.7	12.7
R1		0.0	0.0	0.0	0.0
F0	kN	31.1	29.8	30.9	30.6
Dc	mm	11.1	10.6	10.6	10.8
Kd	kN/mm	80	80	80	80

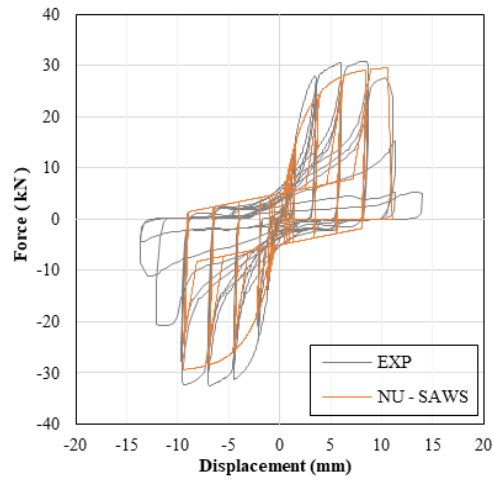
The experimental (EXP) force-displacement curves for each steel connection and the numerical model curves (NU) are shown in Figure 3.6. Relevant sections of the numerical analysis procedure used are provided in Appendix A.



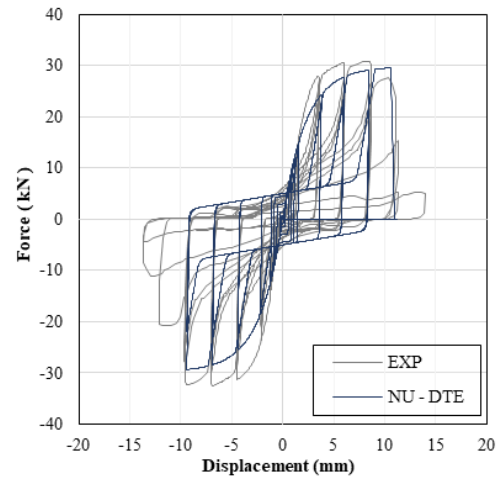
a) 4SC1-SAWS



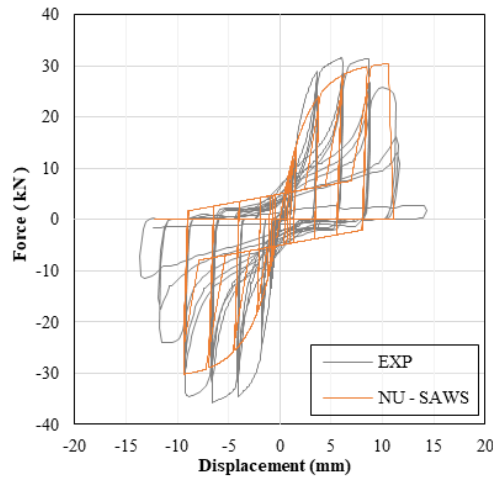
b) 4SC1-DTE



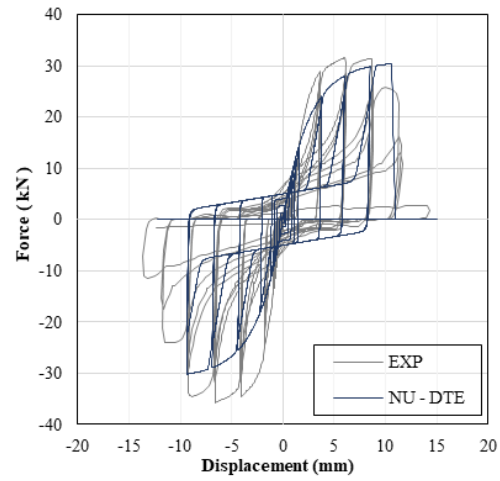
c) 4SC2-SAWS



d) 4SC2-DTE



e) 4SC3-SAWS



f) 4SC3-DTE

Figure 3.6. Steel Connections- Experimental and Numerical Hysteresis Curves

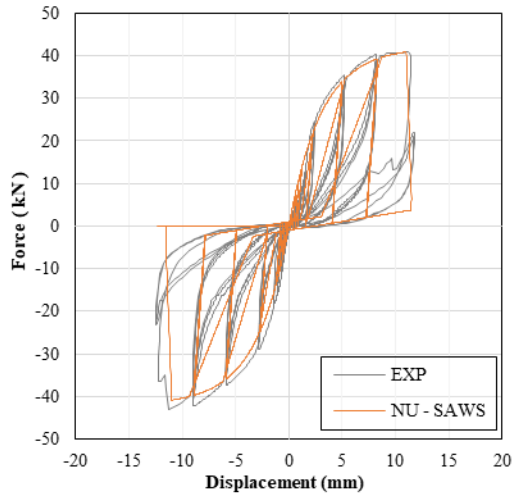
Similarly, the parameters used to calibrate the SMA connection using the SAWS and DTE spring models are provided in Table 3.5 and Table 3.6, respectively. These tables also include the average parameters for each spring model group. The hysteresis curves are provided in Figure 3.7.

Table 3.5. SMA Connections - SAWS Model Parameters

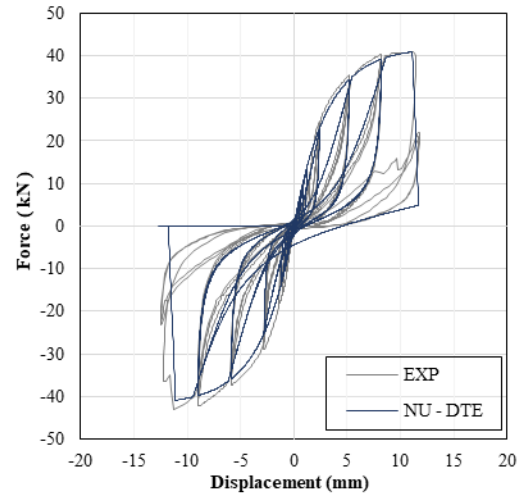
Parameter		4NC1	4NC2	4NC3	4NC_{avg}
F0	kN	42.0	43.0	47.0	43.9
FI	kN	1.0	1.0	1.0	1.0
DU	mm	11.0	11.2	12.1	11.4
S0	kN/mm	13.8	12.5	9.0	11.7
R1		0.0	0.0	0.0	0.0
R2		-5.0	-5.0	-8.0	-5.8
R3		2.5	3.0	3.0	2.8
R4		0.03	0.03	0.05	0.036
alpha		0.7	0.7	0.7	0.7
beta		1.05	1.05	1.05	1.05

Table 3.6. SMA Connections - Dowel-Type Model Parameters

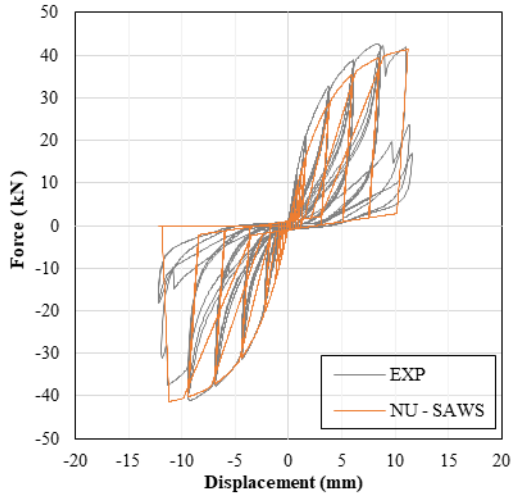
Parameter		4NC1	4NC2	4NC3	4NC_{avg}
Fi	kN	1.0	1.0	1.0	1.0
Kp		2.7	1.5	1.2	1.7
Ru		10.0	10.0	10.0	10.0
c		1.1	1.1	1.1	1.1
beta		1.02	1.02	1.02	1.02
gamma		1.05	1.05	1.05	1.05
eta		0.00075	0.00075	0.00075	0.00075
Dy	mm	1.8	2.4	2.8	2.4
alpha_p		0.6	0.75	0.6	0.65
alpha_u		0.05	0.05	0.05	0.05
alpha_r		0.35	0.35	0.35	0.35
K0	kN/mm	13.8	12.5	9.0	11.7
R1		0.0	0.0	0.0	0.0
F0	kN	42.0	43.0	47.0	43.9
Dc	mm	11.0	11.2	12.1	11.4
Kd	kN/mm	60	60	60	60



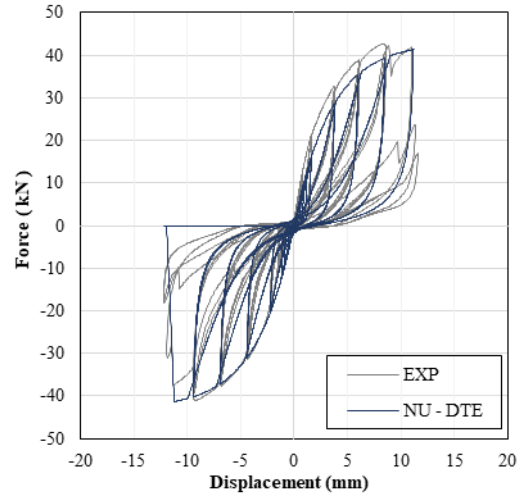
a) 4NC1-SAWS



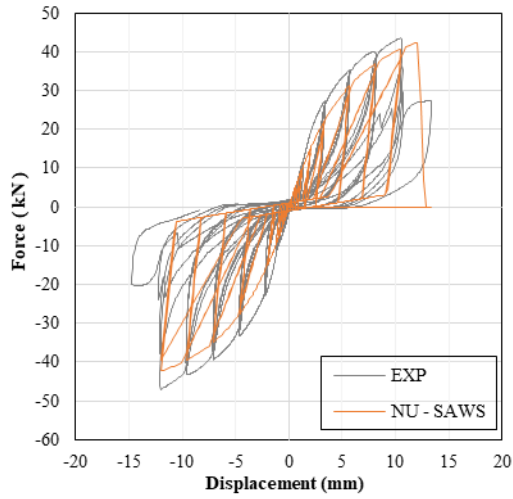
b) 4NC1-DTE



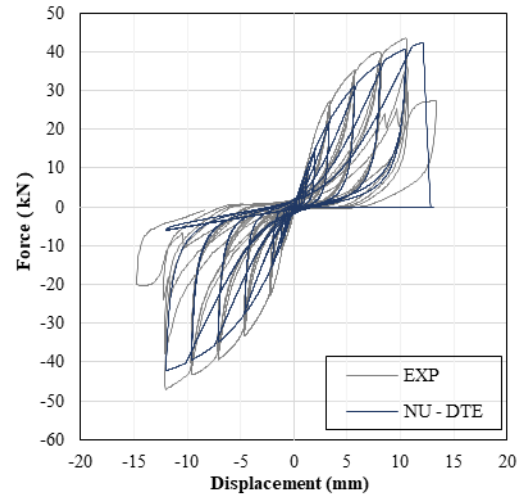
c) 4NC2-SAWS



d) 4NC2-DTE



e) 4NC3-SAWS

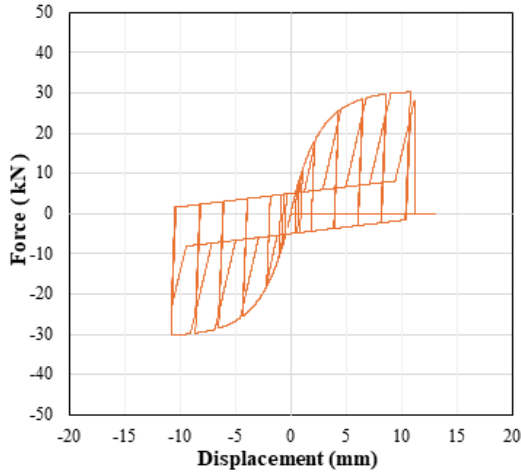


f) 4NC3-DTE

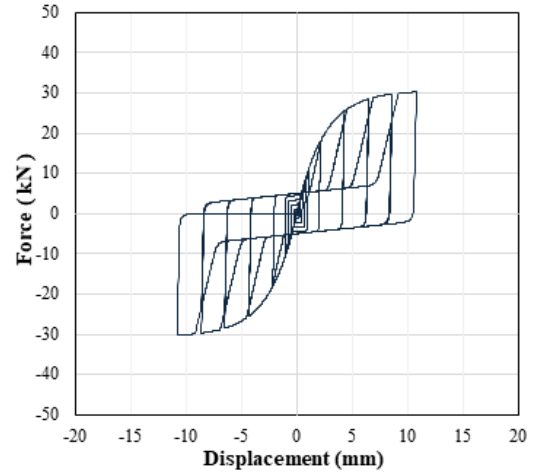
Figure 3.7. SMA Connections- Experimental and Numerical Hysteresis Curves

Overall, the connection-level models calibrated above show good agreement with the experimental loops reported in the reference experiment. These numerical connections were loaded cyclically and exhibited stable hysteretic behaviour, with both materials capturing the post-yielding behaviour. However, after yielding, the steel connections increased gradually and remained steady for the last loops, while the SMA connections increased until peaking at failure. This behaviour aligns closely with the parameters retrieved from the reference experiment and adequately captures their hysteresis curves.

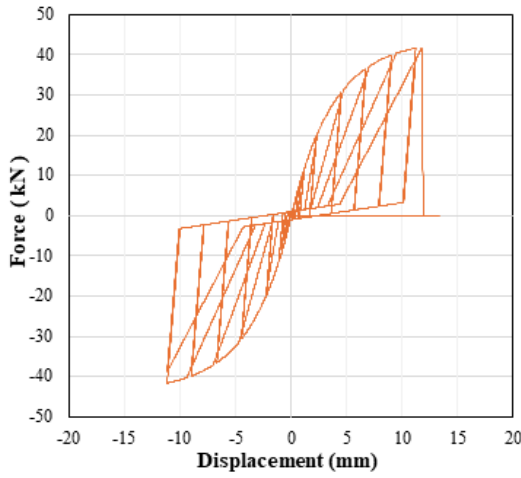
The numerical curves also effectively describe the permanent deformation observed in the unloading branches and stiffness degradation during reloading, which aligns with the cyclic connection behaviour described by Cl  roux et al. (2024). The unloading branches in the steel connections exhibit permanent deformation, while the SMA unloading branches show pinching at the origin, resulting in self-centering behaviour. The numerical behaviour using both SAWS and DTE spring models adequately captures their pinching effect, including their slope and force intercept. However, the DTE spring model captured the slope transitions better than the SAWS spring model. In addition, the hysteresis loops of the steel connections demonstrate greater energy dissipation compared to the SMA connections, which exhibited narrower loops. Figure 3.8 shows the hysteresis curve from the connection-level models with average parameters.



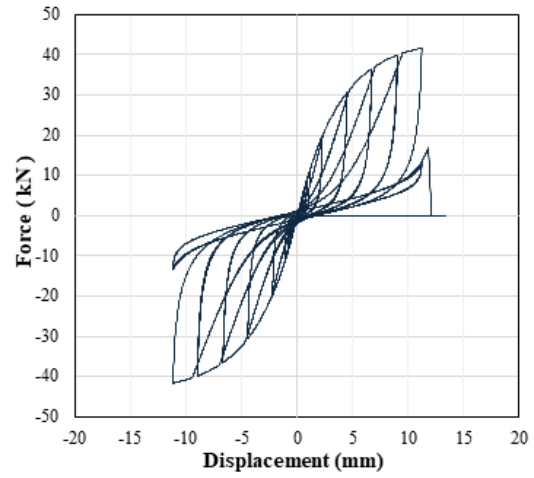
a) 4SC_{avg}-SAWS



b) 4SC_{avg}-DTE



c) 4NC_{avg}-SAWS



d) 4NC_{avg}-DTE

Figure 3.8. Average Numerical Hysteresis Curves

3.3.2 Strong and Weak Connections

The brace-level models incorporated a linear elastic element (111) to represent the diagonal timber brace and two identical nonlinear connection-level models (hinges) to simulate the connections (11). This approach assumed symmetrical behaviour for both connections, as illustrated in Figure 3.9.

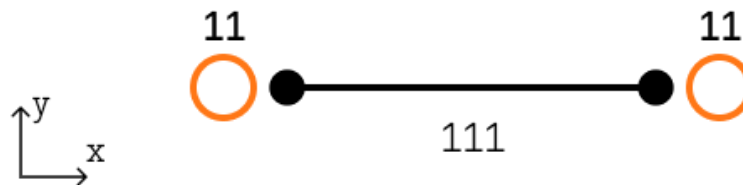


Figure 3.9. Schematic OpenSees - Conventional Brace-Level Model

However, the two connections within a single brace frequently undergo substantially different levels of deformation (Baird et al., 2024; Popovski, 2000, 2004; Popovski et al., 2003). This behaviour will cause one of the brace connections to undergo greater initial deformations and wood crushing. When non-linear deformations begin in one connection, the reduced stiffness of that connection causes an increased deformation demand, ultimately leading to the failure of that specific brace connection (Popovski, 2000; Popovski et al., 2003). The brace that experiences higher deformation is referred to as the weaker connection, and the other brace that has less non-linear deformation is referred to as the stronger connection (Popovski, 2000; Popovski et al., 2003). Given the natural variability of wood as a structural material, predicting which connection will be weak or strong is difficult. However, the locations of the weaker and stronger connections do not affect the overall brace-level and can be used interchangeably in the model. In addition, because the reference experiment by Cl  roux et al. (2024) focused on connection-level testing, the connection was subjected to the highest amount of deformation possible, capturing only the behaviour of a weaker connection.

Based on this information, the modelling approach with two identical connections would most likely overestimate the energy dissipation and ductility of the brace (Chen & Popovski, 2020). The spring theory presented by Chen and Popovski (2020) proposes a relationship between the connection ductility and the brace-level ductility by developing models that accurately capture the behaviour of the stronger connections from previous experimental results and determined that the brace assembly comprises a timber element with a stiffness, K_b , and two connections with stiffness, K_c , linked in series (Figure 3.10).

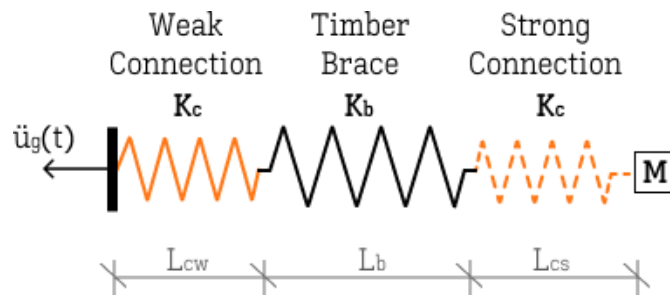


*Figure 3.10. Spring Theory for BTF Systems
Adapted from Chen and Popovski (2020)*

Chen and Popovski (2020) also noted that the weaker connection has a lower yield strength than the stronger connection and absorbs nearly all the nonlinear deformations, resulting in the brace assembly's yield strength, F_{bay} , governed by the weaker connection, F_{cwy} . This implies that the yield strength of the weaker connection would be the same as that of the stronger connection, and after the connection has

yielded, the stronger connection will no longer contribute to the resistance of the entire brace system. The yield displacement, Δ_{bay} , for the spring model presented by Chen and Popovski (2020) is shown in the equation (3.1). Its equivalent stiffness formulation for the linear elastic behaviour, $K_{e,y}$, is given in equation (3.2), where the weaker connection stiffness is K_{cw} and the brace member stiffness is K_b . Implementing this spring approach, the revised brace-level model uses one linear elastic element (111) for the wood member, a nonlinear spring (11) for the weaker connection, and a second nonlinear spring (12) for the stronger connection.

$$\Delta_{bay} = 2 \frac{F_{cwy}}{K_c} + \frac{F_{cwy}}{K_b} \quad (3.1)$$

$$K_{e,y} = \frac{1}{\frac{2}{K_{cw}} + \frac{1}{K_b}} \quad (3.2)$$

For plastic behaviour, the ultimate brace displacement (Δ_{bau}) formulation by Chen and Popovski (2020) is provided in equation (3.3). This relationship combines the ultimate displacement of the weaker connection (Δ_{cwu}), and the elastic yield displacement of the stronger connection and timber brace. Once the weaker connection yields, the stronger connection does not contribute to the deformation of the brace, and it is treated as a rigid element. The plastic stiffness is given by equation (3.4).

$$\Delta_{bau} = \Delta_{cwu} + \frac{F_{cwy}}{K_c} + \frac{F_{cwy}}{K_b}. \quad (3.3)$$

$$K_{e,p} = \frac{1}{\frac{1}{K_{cw}} + \frac{1}{K_b}} \quad (3.4)$$

3.3.3 Brace-Level Model

After defining the behaviour of the stronger connection based on the calibrated weaker connection hinges, the brace-level model can be established as a system consisting of a weaker connection, a stronger connection, and a timber brace. The weaker connection is modelled as a nonlinear spring element (11), while the stronger connection is also represented by a nonlinear spring element (12) derived from the elastic response of element (11). The timber brace is modelled as a linear elastic element (111). The revised brace-level model is shown in Figure 3.11.

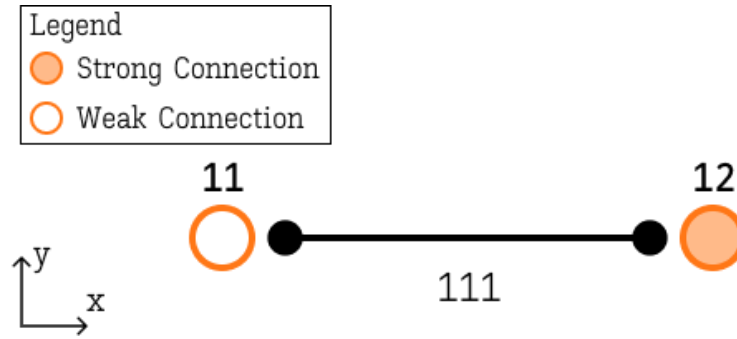


Figure 3.11. Schematic Weaker-Stronger Brace-Level Model in OpenSees

This brace-level modelling approach, developed in OpenSees, is comparable to the equivalent nonlinear connector element proposed by Chen and Popovski (2020) to create an analytical model that captures the behaviour of a brace with a weaker and a stronger connection. The equivalent nonlinear connector element is part of various multistorey building prototypes developed to determine the system ductility. Chen and Popovski (2020) reported differences of less than ± 5 percent when comparing the system ductility from the numerical models with the predicted values from the proposed connection and system ductility equations.

The model presented in Figure 3.11 was then subjected to a displacement-control loading protocol. The brace system was fixed at one end while the other was subjected to uniaxial loading. The location of the weaker and stronger connections (11 and 12) was arbitrary, as this has no impact on the system-level behaviour. The timber member (111) represents a 20f-EX glulam element with a cross-section of 127 x 140 mm, replicating the reference experiment conditions, and an assumed length of 4,000 mm. The elastic properties of the brace were modelled with a modulus of elasticity of 10,300 MPa, in accordance with CSA (2019) standards. The brace-level system models are illustrated in Figure 3.12, with a corresponding summary of the model configurations provided in Table 3.7.

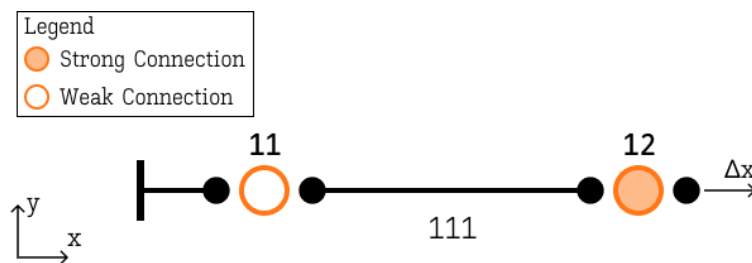


Figure 3.12: Schematic Brace-Level Model in OpenSees

Table 3.7: OpenSees Brace-Level (BL) Models Schedule

Connections	Brace-Level (BL) Models	
	OS-SAWS	OS-DTE
Steel		
4SC1	BL-4SC1-SAWS	BL-4SC1-DTE
4SC2	BL-4SC2-SAWS	BL-4SC2-DTE
4SC3	BL-4SC3-SAWS	BL-4SC3-DTE
4SC _{avg}	BL-4SC _{avg} -SAWS	BL-4SC _{avg} -DTE
SMA		
4NC1	BL-4NC1-SAWS	BL-4NC1-DTE
4NC2	BL-4NC2-SAWS	BL-4NC2-DTE
4NC3	BL-4NC3-SAWS	BL-4NC3-DTE
4NC _{avg}	BL-4NC _{avg} -SAWS	BL-4NC _{avg} -DTE

The yield and ultimate displacements obtained using the spring theory relationships (ST) were compared with those from the numerical model in OpenSees (OS) to evaluate the accuracy of the brace-level models. The ST displacements were calculated based on the weaker connection properties listed in Table 3.2 and using equations (3.1) and (3.3). The stiffness of the timber brace, K_b , was calculated to be 45 kN/mm. Table 3.8 presents the yield and ultimate displacements from both ST and OS models (SAWS and DTE) and the differences between the theoretical and numerical results.

Table 3.8. Theoretical and Numerical Displacements for Brace-Level Models

Connection	ST		OS-SAWS				OS -DTE			
	D _{ba,y}	D _{ba,u}	D _{ba,y}		D _{ba,u}		D _{ba,y}	%	D _{ba,u}	%
Steel										
BL-4SC1	3.36	13.00	3.67	9.3%	13.18	1.4%	3.66	9.0%	13.18	1.4%
BL-4SC2	2.34	11.92	2.02	-13.7%	12.13	1.8%	2.02	-13.7%	12.17	2.1%
BL-4SC3	2.61	12.05	2.14	-17.9%	12.20	1.2%	2.15	-17.6%	12.20	1.2%
BL-4SC _{avg}	2.79	12.36	2.54	-8.8%	12.42	0.4%	2.55	-8.5%	12.46	0.8%
SMA										
BL-4NC1	3.94	13.23	4.05	2.7%	13.64	3.1%	3.98	0.9%	13.64	3.1%
BL-4NC2	4.61	13.79	5.04	9.3%	14.40	4.5%	5.04	9.3%	14.44	4.7%
BL-4NC3	5.67	15.19	5.48	-3.4%	15.55	2.3%	5.51	-2.8%	15.66	3.1%
BL-4NC _{avg}	4.62	13.98	4.85	5.0%	14.62	4.6%	4.85	5.0%	14.58	4.3%

Comparison between the yield ($D_{ba,y}$) and ultimate ($D_{ba,u}$) displacement results show strong overall agreement. Most cases demonstrate a high degree of alignment with percentage differences remaining below 5%; however, the yield displacement results for all numerical models are highly variable compared to the predicted displacement using the ST method. In addition, the ultimate displacements of the brace-

level models with steel connections matched the ST values within 2% while the yield displacements were observed to range 0.82 – 1.09 times those obtained using the ST method. Similarly, the brace-level models with SMA connections showed good agreement with ultimate displacements within 5% of the ST values, and yield displacements ranging from 0.97 to 1.09 times those predicted by the ST method. However, the 4NC2 model stands out with an overestimated 9.3%, highlighting a slight deviation.

3.4 Pushover Analysis

A nonlinear static pushover analysis procedure is a seismic performance assessment method that applies a controlled lateral displacement until the structure fails. This method provides insight into the ultimate displacement and failure mechanisms of the system of interest under lateral loads. In this study, single-storey frame-level models incorporating steel and SMA connection-level models were developed in OpenSees. These frame-level models were subjected to a pushover procedure using a node control protocol to evaluate their nonlinear behaviour, ultimate displacement, and self-centering potential.

The numerical pushover analysis was carried out in both monotonic and cyclic stages. The monotonic procedure was performed to determine the ultimate displacement of the frames, and the results were compared with theoretical predictions based on the spring theory developed by Chen and Popovski (2020). This comparison aimed to validate the frame-level models by quantifying the difference between the theoretical displacement values and the OpenSees models. At the same time, a cyclic pushover analysis was conducted to investigate the hysteresis loops, especially the unloading branches, to compare the permanent deformation and energy dissipation capacity of steel and SMA connection. The frame-level models were subjected to cyclic displacement-controlled loading following the experimental protocol used for the connection-level models.

3.4.1 Frame Model and Analysis

The frame-level models were developed in OpenSees using elastic beam-column elements for the timber members and zeroLength elements for the brace connections. The diagonal members were modelled based on the brace-level models presented in Section 3.3.3. In total, 16 frame-level numerical models were developed. Eight versions created for each fastener material (steel and SMA), with four frames using the DTE connection model and four using the SAWS connection model (refer to Figure 3.7). These numerical models were subjected to loading in the horizontal direction using a displacement control node as shown in Figure 3.13.

The timber elements consist of 20f-EX glulam members with a cross section of 127 x 140 mm and an elastic modulus of 10 300 MPa. The frame was assumed to support a gravity load of 53 kN, representing the weight of the building, to consider P-Delta effects. The nodes of the frame-level model were allowed to rotate freely, ensuring that the timber elements remained elastic while the plastic hinges captured the nonlinear deformation of the structure. The columns were pinned at the ground level. It was assumed that all other connections were sufficiently strong to behave elastically until failure occurred in the brace connections.

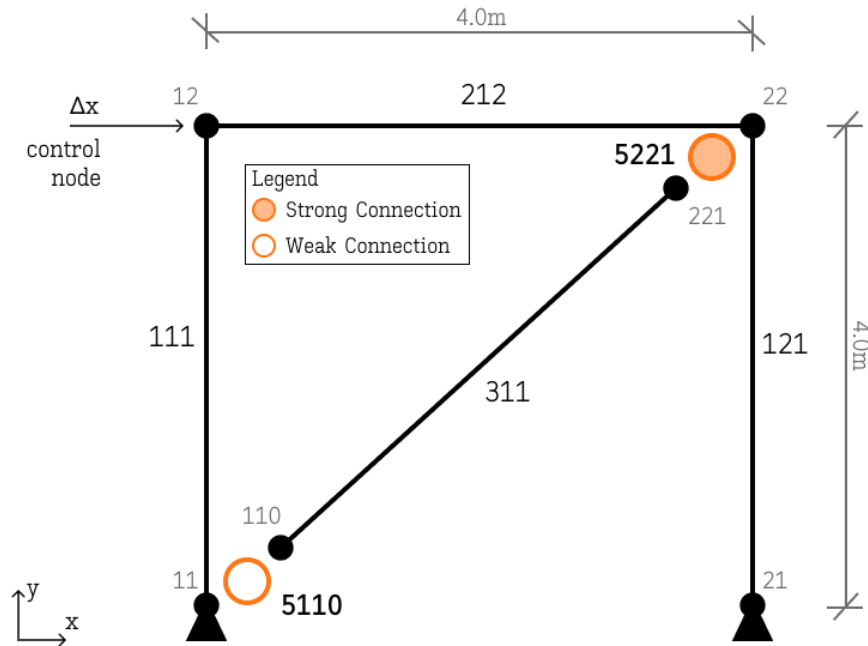


Figure 3.13. Schematic Diagram of Frame Model for Pushover Analysis

To perform the pushover analysis, a gravity analysis was first conducted on the frame to establish the initial conditions that the structure supported before the application of lateral loads. Following the gravity analysis, a lateral load pattern was defined by applying incremental target displacements using a displacement control procedure. This approach (provided in Appendix A) allowed the internal forces and deformations within the structure to be calculated at each step, ensuring accurate results of the behaviour of the frame as the lateral displacement increased. The analysis continued until the brace connections failed, resulting in frame instability and collapse. The structural response was quantified by plotting the base shear versus the displacement at node 22, providing valuable insights into the nonlinear performance and ultimate capacity of the frame under lateral loading.

3.4.2 Monotonic Pushover

The frame models were pushed under lateral load patterns until failure was reached. Displacement at the top of the frame and the corresponding shear force at the base of the models were used to define the force-displacement relationships. This monotonic loading analysis was studied to compare the ultimate displacement between theoretical equations and numerical models, similar to the comparison presented in Section 3.3.3. Figure 3.14 illustrates the monotonic pushover curves for both the steel and SMA connections using the SAWS and DTE spring models.

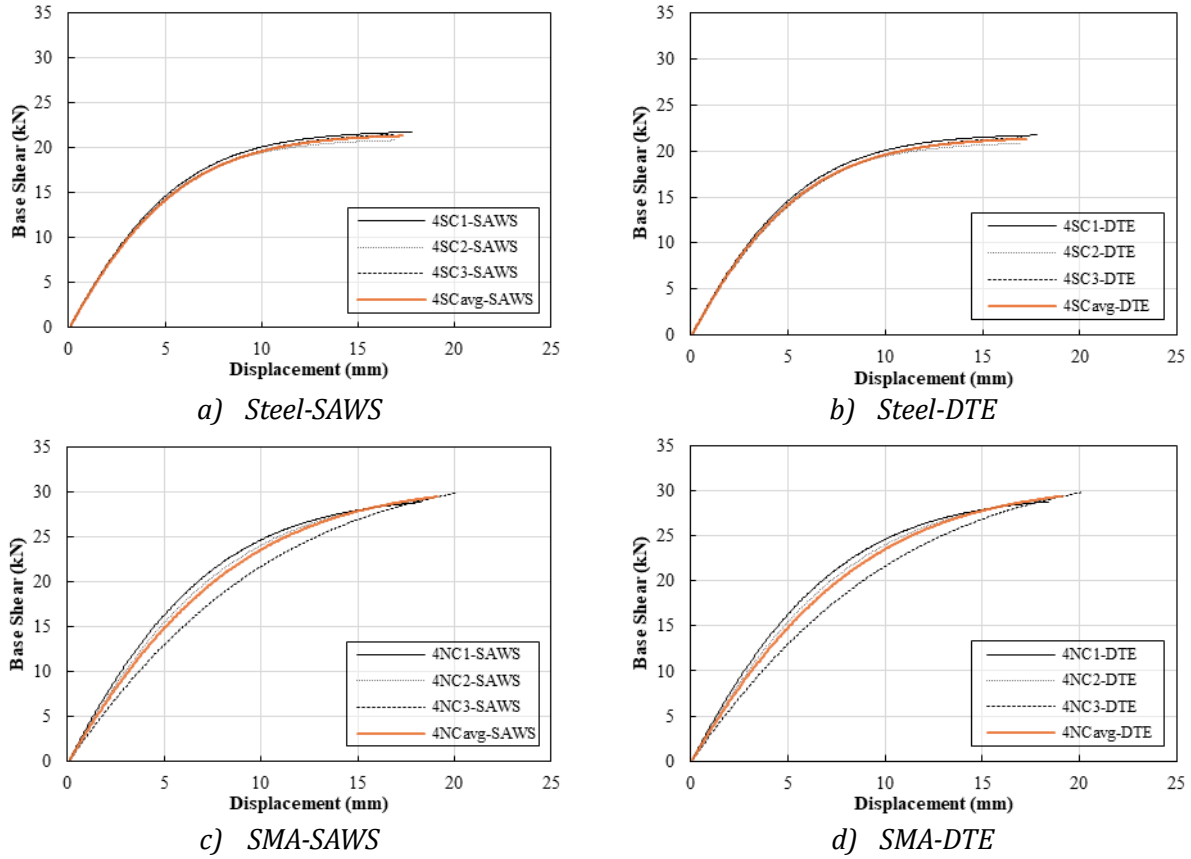


Figure 3.14. Monotonic Pushover Results of Frame-Level Models

The ultimate displacement, $D_{bf,u}$, for the DTE and SAWS spring models was obtained from the response curves of the OpenSees models. The ultimate displacements derived using the ST method were compared with the numerical values obtained from OpenSees (OS) in order to evaluate the frame-level models. The ST displacements were calculated based on the horizontal component of the brace ultimate displacement, $D_{ba,u}$, (reported in Table 3.8), using the angle of the diagonal brace to the horizontal plane. A detailed comparison of the ultimate displacements from the theoretical (ST) and numerical (OS) models, along with the percentage differences, is presented in Table 3.9.

Table 3.9. Theoretical and Numerical Displacements for Frame-Level Models

	ST	OS-DTE		OS-SAWS	
Model	D _{bf,u}	D _{bf,u}	%	D _{bf,u}	%
Steel					
4SC1	18.38	17.82	-3.0%	17.81	-3.1%
4SC2	16.86	16.92	0.4%	16.91	0.3%
4SC3	17.05	17.01	-0.2%	16.87	-1.0%
4SCavg	17.49	17.27	-1.2%	17.26	-1.3%
SMA					
4NC1	18.72	18.42	-1.6%	18.27	-2.4%
4NC2	19.50	18.67	-4.2%	18.62	-4.5%
4NC3	21.49	20.47	-4.7%	20.51	-4.5%
4NCavg	19.77	19.12	-3.3%	19.11	-3.3%

The comparison of ultimate displacements derived from the spring theory (ST) method and numerical OpenSees (OS) models shows a high degree of consistency, with percentage differences remaining below 5% for both the DTE and SAWS spring models. The frame-level displacement predictions for both materials exhibit substantial similarity but generally demonstrate slight underestimation when compared to values obtained using ductility-based equations. Furthermore, the displacements for the steel models with DTE spring models show a closer alignment with the numerical results than the SAWS spring models.

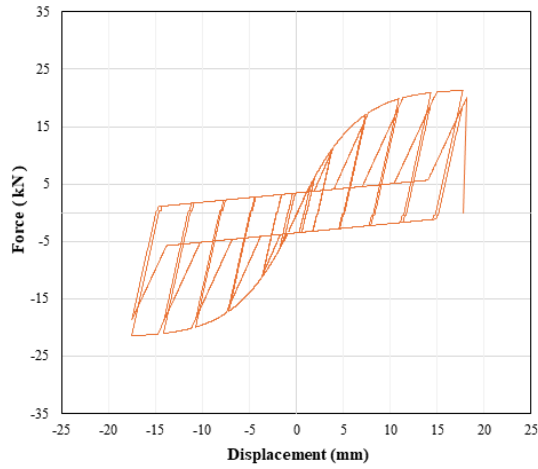
3.4.3 Cyclic Pushover

To assess the cyclic behaviour and self-centering potential of SMA dowels in a frame-level analysis, the frame models were subjected to dynamic displacements simulating earthquake loading. The cyclic pushover analysis used frame-level models incorporating the average parameters for the SAWS and DTE materials only. The loading protocol for the cyclic analysis defined by Cl  roux et al. (2024) followed the modified Test Method B ASTM E2126 (2019) protocol. Details of the cyclic loading protocol used in the analysis are provided in Table 3.10.

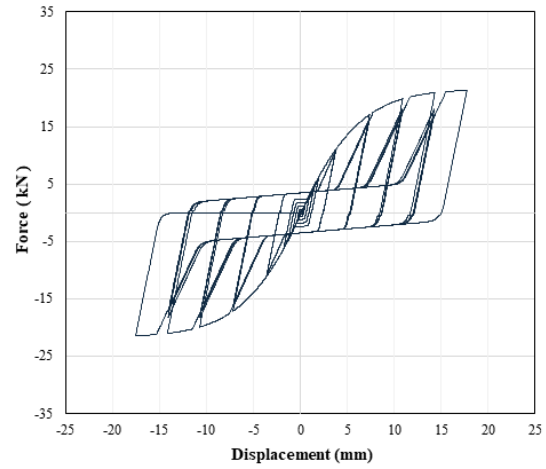
Table 3.10. Cyclic Pushover Analysis - Loading Protocol

Step	Cycle	Amplitude	4SC	4NC
			Disp. (mm)	Disp. (mm)
1	1	1.25%	0.22	0.24
2	1	2.5%	0.43	0.48
3	1	5.0%	0.86	0.96
4	1	7.5%	1.30	1.43
5	1	10.0%	1.73	1.91
6	3	20.0%	3.45	3.82
7	3	40.0%	6.91	7.65
8	3	60.0%	10.4	11.5
9	3	80.0%	13.8	15.3
10	3	100.0%	17.3	19.1
11	3	120.0%	20.7	22.9

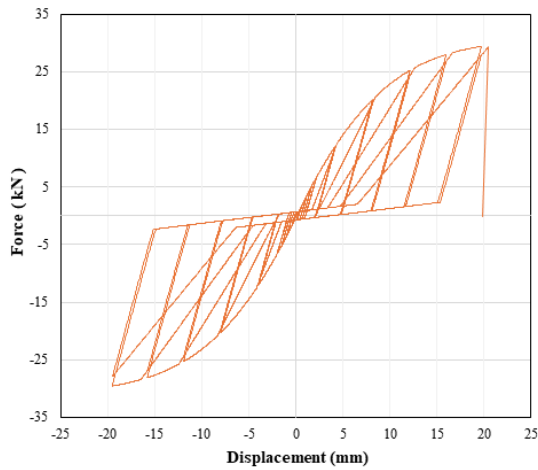
The hysteric force-displacement responses for the frame-level models with the average connection parameters are shown in Figure 3.15. The frame-level curves exhibit a strong resemblance to the weaker connection-level models illustrated in Figure 3.8, demonstrating that the plastic behaviour of the frame is mainly governed by the weaker connections. These connections provide the required ductility and energy dissipation to enable the frame to withstand lateral forces. Frame-level models with steel and SMA spring models exhibited stable hysteretic behaviour under cyclic loading, similar to the numerical connection-level model curves.



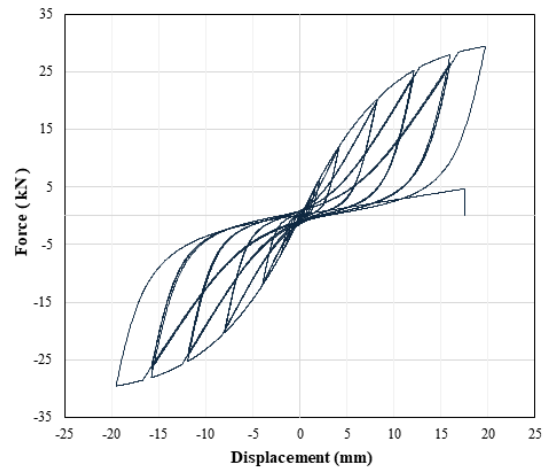
a) Steel-SAWS



b) Steel-DTE



c) SMA-DTE



d) SMA-DTE

Figure 3.15. Pushover Analysis - Hysteresis Curves

The force-displacement curves for frame-level models exhibit post-yield hardening behaviour, especially in the SMA models. The envelope curves for the DTE and SAWS models are similar, with comparable ultimate displacements. However, the SMA spring models achieve a slightly larger average ultimate displacement of 19.1 mm compared to 17.3 mm for the steel spring models. While both material models show a pinched response, the unloading branches of the hysteresis curves display significant differences. In the steel connection model, the unloading branches clearly exhibit permanent deformation. In contrast, the SMA connection model demonstrates self-centering behaviour, as evidenced by the small deformation in the unloading branches. The SMA superelastic properties allow it to recover its original shape upon unloading, which explains the absence of significant pinching and the close return of the displacement path to the origin. The unloading behaviour aligns with the conclusions from the connection-level study by Cl  roux et al. (2024), reinforcing the premise that SMA connections contribute to reducing residual displacements.

3.5 Time History Analysis

Numerical models were also developed to evaluate the seismic performance of single-storey and multi-storey BTFs using nonlinear time history analysis (NTHA) in OpenSees. This study aims to investigate the structural response of braced frames with steel and SMA dowel-type connections under earthquake ground motions. The single-storey frame-level model developed earlier was subjected to dynamic analysis to study the self-centering ability of the structure. Meanwhile, the multi-storey frame-level models expand on these findings by assessing the impact of stiffness and bracing configurations.

The numerical procedure began with a modal analysis to evaluate the dynamic properties of the modelled frames through eigenvalue analysis. Following this, an NTHA was conducted using the Newmark-Beta method to simulate the structural response under seismic ground motions. The numerical procedure has been provided in Appendix A. A total of fifteen ground motion records were selected based on site-specific conditions and literature for Ottawa. To ensure compatibility with the seismic demand, these records were scaled to align with both the target response spectrum and the recorded design spectra at the fundamental period of each frame model. The target design spectrum used in this study corresponds to the National Building Code of Canada (NBCC, 2020) seismic provisions.

The NTHA results offer valuable insights into the dynamic response of braced timber frames with steel and SMA connections, specifically in terms of peak drift, residual drift, and peak floor acceleration across different storeys. While SMA-connected frames generally experience greater peak drifts, they exhibit a notable advantage in self-centering behaviour, significantly reducing residual drift compared to steel-connected frames. Additionally, the multi-storey frame results highlight the impact of bracing configuration on stiffness distribution, demonstrating how variations in bracing arrangement impact the structural response under seismic loading scenarios.

3.5.1 Frame Models

Expanding upon the pushover frame-level models, three building prototypes were developed, including a single-storey, a three-storey, and a four-storey structure. These archetypes represent lateral load-resisting systems for single- and multi-storey braced mass timber buildings and were analyzed using two-dimensional numerical models with a time history analysis method. Consistent with previous models, the frame-level numerical models were developed in OpenSees using elastic-beam elements for the glulam beams, columns, and braces and non-linear spring elements for the brace connections at the diagonal members. The connections between other timber members were modelled as free to rotate to reflect the

expected behaviour of BTFs accurately. This approach assumes that beam-to-column connections are designed to provide sufficient resistance while allowing for necessary displacements. By incorporating this assumption, the numerical model effectively captures the internal force distribution, ensuring that the diagonal braces and their connections primarily govern the lateral resistance of the system. The numerical frame-level models developed in this study are illustrated in Figure 3.16.

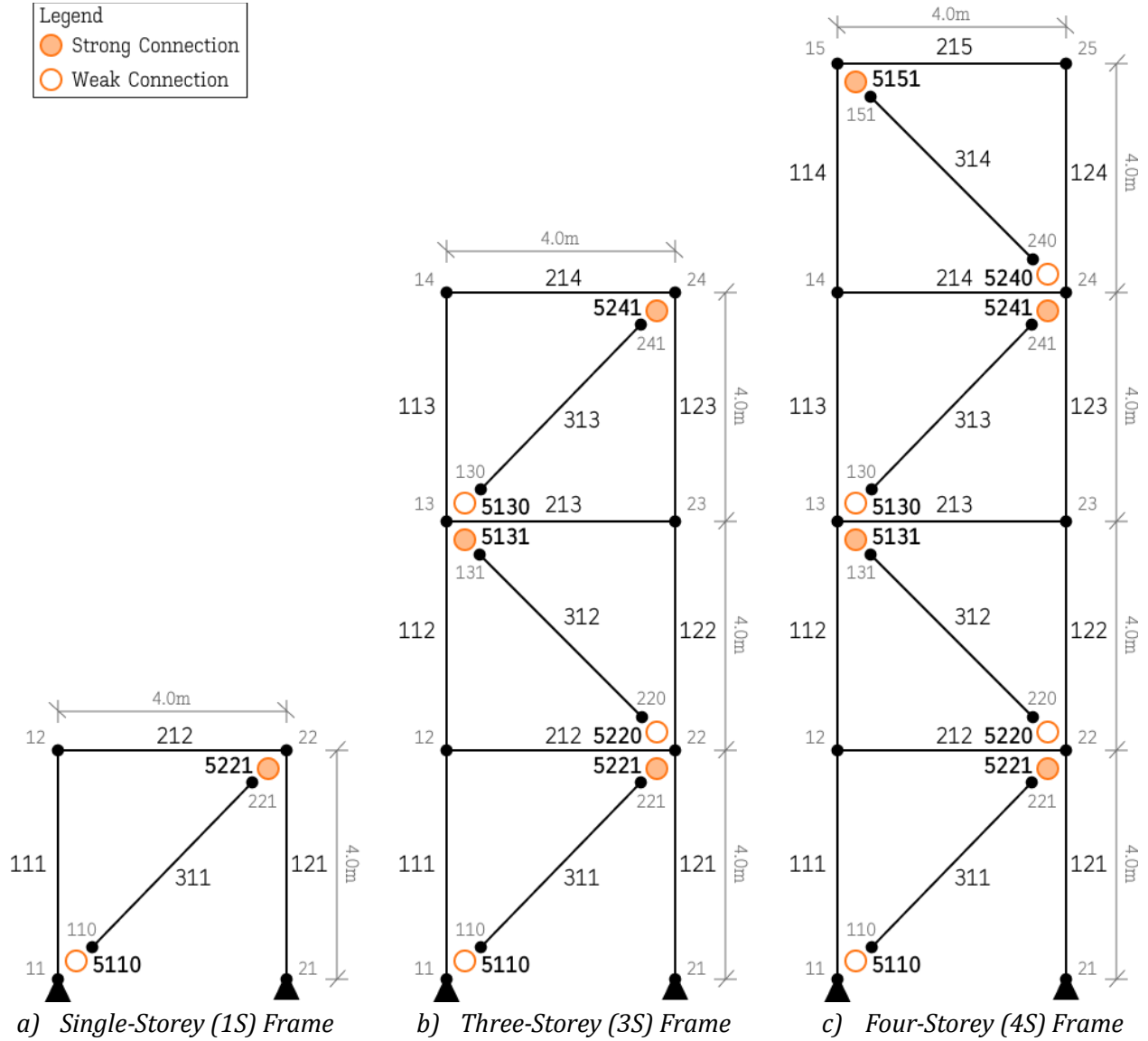


Figure 3.16. Building Frame Numerical Models

After confirming the modelling framework with the single-storey (1S) model, two multi-storey frames were developed to assess how vertical brace layout and overall rigidity influence seismic response. The multi-storey frames consisted of:

- Three-Storey (3S) Frame: Braces were arranged so that the brace at the middle level is in the opposite direction than the other two levels, introducing a change in stiffness along the height. The overall frame stiffness is equal to 0.89 kN/mm.
- Four-Storey (4S) Frame: The added brace results in a uniform stiffness profile. The storey stiffness remains the same; however, the global stiffness reduces to 0.66 kN/mm.

The analysis of these two multi-storey frame cases allows to understand how an asymmetrical versus a uniform stiffness distribution affects the dynamic response of timber braced frames, particularly regarding seismic performance and self-centering capabilities.

The connection spring elements were modelled using zeroLength elements and assigned zeroLength material properties based on the calibrated average parameters for the DTE spring models. Although the SAWS and DTE springs successfully replicated the cyclic hysteresis of the bolt tests, only the DTE design was used in this dynamic analysis. As mentioned in Section 2.5.1, the DTE law is based on the SAWS backbone but substitutes its sudden slope changes with smooth curve transitions, resulting in the same force-deformation envelope while removing the abrupt bend curves that cause extra iterations during unloading and reloading cycles. In this study, trial runs revealed that frames with SMA spring models required several times more calculation time, with no change in peak or residual response metrics, because the solver struggled to converge at each slope break. To keep the parametric study feasible timewise while preserving result fidelity, the NTHA used only the DTE springs, recognizing that their output is equivalent to SAWS but significantly less computationally expensive.

Since these connections were designed to replicate the experimental testing conditions, the diagonal members were specified as 20f-EX SPF glulam sections with dimensions of 127 × 140 mm and an elastic modulus of 10,300 MPa following the experimental setup. The design of the multi-storey frames was based on the assumption of elastic behaviour in the timber connections, with the yield resistance of steel and SMA connections taken as 15.5 kN and 24.1 kN, respectively.

All building archetypes were assumed to be in Ottawa, with loads determined according to the climatic data specified in the NBCC (2020). The gravity loads were calculated based on the most critical load combination, which included 100% of dead loads and 25% of transient loads (live and snow), resulting in floor loads of 13.2 kN/m and a roof load of 10.8 kN, assuming a tributary width of 4 metres. The lateral seismic loads were determined using the Equivalent Static Force Procedure outlined in the NBCC (2020), assuming a Site

Class C. Seismic force modification factors were established based on the reported values by Chen and Popovski (2020), considering limited ductility for braced timber frames.

Once the lateral forces were determined, an iterative design process was conducted following the Engineering Design in Wood (CSA, 2019) to size the columns. Since the brace dimensions and connection capacities were constrained, the column size was adjusted accordingly. The final column design consisted of 175×190 mm 12c-E SPF glulam sections with an elastic modulus of 9,700 MPa. This column size was determined based on the worst-case scenario of the four-storey frame, resulting in an overdesign for the lower-rise building models. The beam sections were assigned the same cross-section dimensions and grade type as the braces.

3.5.2 Methodology

A numerical procedure was developed to determine the structural response as a time variation of the ground acceleration using a nonlinear time history analysis (NTHA) method. The numerical analysis starts with a gravity analysis to establish the initial loading conditions. Following this, a modal analysis is performed to assess the dynamic characteristics of the building frames. An eigenvalue analysis was conducted using a built-in OpenSees function, which computes the eigenvalues by assembling the stiffness and mass matrices based on the defined model. The damping value selected was 3% following recommendations by Popovski (1999).

The modal results are processed to determine dynamic parameters, including natural frequencies, mode shapes, and vibration periods. The first and second vibration modes are specifically used to determine the damping coefficients, which are then incorporated into the numerical model through the Rayleigh damping formulation. Using only the first two modes is justified, as higher mode effects do not significantly influence the structural response. This assumption is further supported by the findings of Chen and Popovski (2021), who demonstrated that the first vibration mode dominates the dynamic response of braced timber frames.

Following the modal analysis, the numerical evaluation proceeded with a time history analysis to assess the structural response over a defined time duration. The computational approach used the Newmark-Beta method, a widely used numerical integration technique for solving dynamic equations of motion. In addition, the analysis requires that a deformation capacity be specified to define a limit that, upon exceedance, will stop the NTHA and mark the analysis as a collapse case. For this purpose, a 0.5% peak interstorey drift (20 mm) was selected slightly above the maximum brace displacement observed in the pushover analysis. This threshold is applied to the deformation of the top node to flag collapse and terminate

the simulation. This approach prevents the model from obtaining unrealistic results in an indefinite convergence loop when structural failure has effectively occurred, eliminating the need for extensive post-processing to identify collapsed cases. The collapse criteria are defined based on local story drifts and roof drift in either direction to capture different failure modes. Finally, the algorithm attempts various solution routines to enhance numerical convergence and complete the ground motion simulation. This adaptive approach improves the robustness of the analysis.

The analysis was designed to incorporate ground motion records by reading data from externally selected files, chosen based on the specific site location. The building frames were designed for a site in Ottawa for a Site Class C, corresponding to a shear wave velocity (V_{s30}) ranging from 360 to 760 m/s. Ottawa is situated within the Western Quebec Seismic Zone (WQSZ), a region of low to moderate seismic activity in eastern Canada, where strike-slip earthquakes are the predominant seismic events (Rimando & Benn, 2005). In this region, significant earthquakes are those with magnitudes exceeding M4.5, with historical records indicating no events greater than M6.5 (S. Ma & Eaton, 2007). Most recorded earthquakes in the area are of M3 magnitude. According to a study by Atkinson (2009) on time history spectra for eastern Canada, M6 seismic events typically occur at distances ranging from 10 to 30 km for short periods (<1.0 sec). The design spectrum for Ottawa (City Hall) corresponds to a 2% probability of exceedance in 50 years and is characterized by a peak ground acceleration (PGA) of 0.207 m/s^2 and a peak ground velocity (PGV) of 0.147 m/s . The target response spectrum used in this study is illustrated in Figure 3.17.

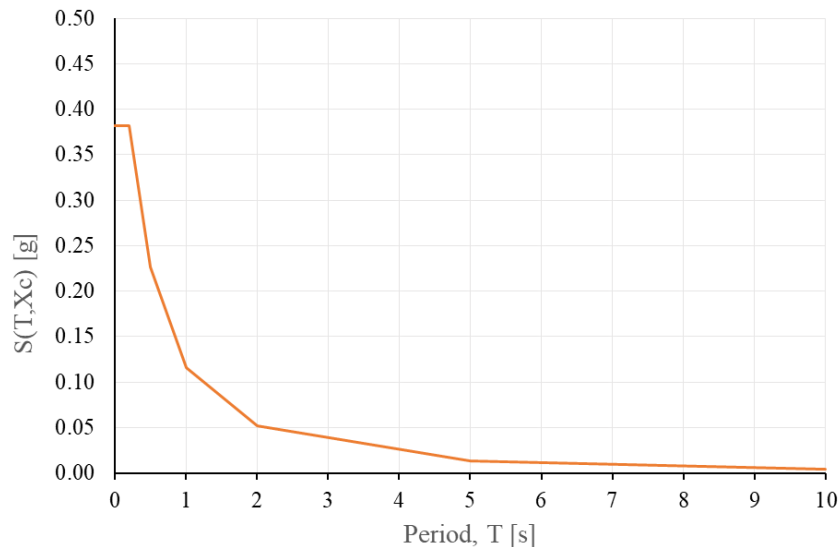


*Figure 3.17. Design Spectrum, Ottawa - Site Class C
Retrieved from NBCC (2020)*

The selection of ground motion records was based on the earthquake characteristics specific to the site location, as previously described. Two earthquake scenarios were considered: (1) events with magnitudes ranging from M4.5 to below M6, and (2) M6 events occurring at rupture distances between 10 and 30 km, both representing strike-slip seismic activity. A total of 15 ground motion records were chosen, with 10 corresponding to the first scenario. The ground motion data was sourced from the Pacific Earthquake Engineering Research Center (PEER) database (Ancheta et al., 2014). However, due to the limited availability of seismic records for Eastern North America, the selected motions were primarily taken from active tectonic zones in Western North America. For this study, the horizontal acceleration components of the seismic records were extracted and utilized for time history analysis. To ensure compatibility with the site-specific seismic demand, the selected ground motions were scaled to match the target design spectrum at the fundamental period of each building frame model. The details of the ground motion time histories and the applied scaling factors are presented in Table 3.11 and Table 3.12, respectively.

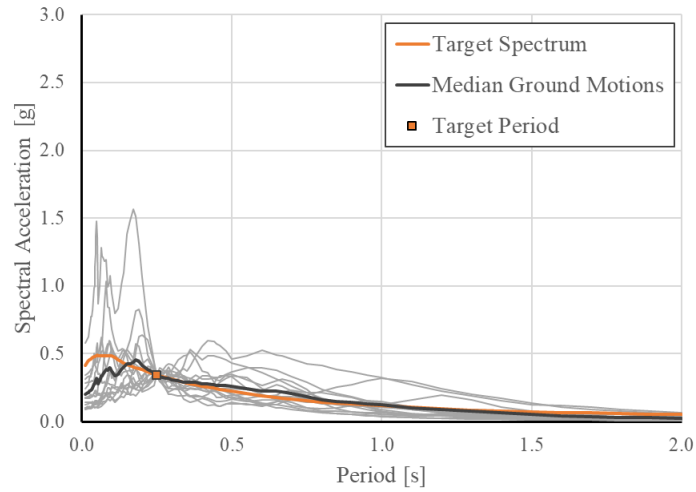
Table 3.11. Earthquake Records

No.	ID	Year	Event Name	Station	M	R
Eq1	152	1979	Coyote Lake	SJB Overpass Bent 3 g.l.	5.74	20.67
Eq2	222	1980	Livermore-02	Livermore - Morgan Terr Park	5.42	14.12
Eq3	234	1980	Mammoth Lakes-02	Long Valley Dam	5.69	16.88
Eq4	242	1980	Mammoth Lakes-04	Long Valley Dam	5.70	14.38
Eq5	1099	1994	Double Springs	Woodfords	5.90	12.84
Eq6	4260	1972	Ancona-09_ Italy	Ancona-Rocca	4.70	10.10
Eq7	4331	1990	Sicilia-Orientale_ Italy	Sortino	5.60	26.93
Eq8	9009	2005	14151344	Anza_ Mitchell Rd.	5.20	14.19
Eq9	9449	2005	21465580	Truckee - I80/Truckee River Bridge	4.77	10.07
Eq10	9838	2007	51182810	Convict Creek	4.60	13.42
Eq11	450	1984	Morgan Hill	Corralitos	6.19	23.24
Eq12	2709	1999	Chi-Chi_ Taiwan-04	CHY035	6.20	25.06
Eq13	4284	1978	Basso Tirreno_ Italy	Naso	6.00	19.59
Eq14	6057	1992	Big Bear-01	Highland Fire Station	6.46	26.47
Eq15	6878	1992	Joshua Tree_ CA	North Palm Springs Fire Sta #36	6.10	21.97

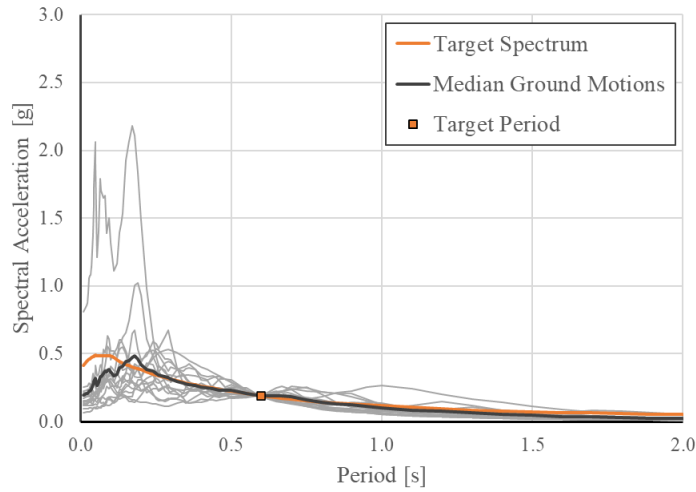
Table 3.12. Ground Motion Scale Factors

No.	1S	3S	4S
Period [s]	0.25	0.62	0.81
Eq1	2.33	1.10	1.45
Eq2	0.93	1.09	1.36
Eq3	1.60	1.06	1.64
Eq4	1.30	1.27	0.89
Eq5	1.46	1.16	1.26
Eq6	1.09	1.53	1.50
Eq7	3.23	1.17	0.76
Eq8	0.97	0.95	2.24
Eq9	2.25	1.21	1.26
Eq10	0.84	1.31	1.13
Eq11	1.26	1.17	0.91
Eq12	1.21	1.01	0.66
Eq13	0.67	1.12	0.93
Eq14	0.89	1.39	1.92
Eq15	0.58	0.86	1.39

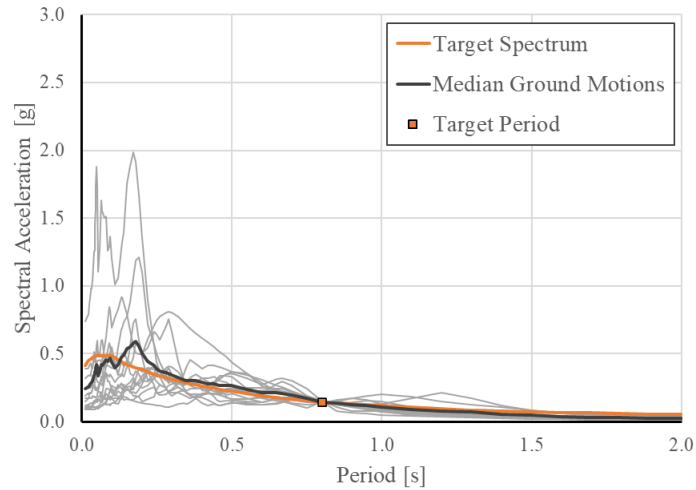
Figure 3.18 presents the scaled acceleration response spectra for all selected ground motion records corresponding to each building model. The ground motions were scaled to ensure that their response spectra align with the target design spectrum at the fundamental period of each building frame. The median response spectrum is also included to represent the trend of the scaled records.



a) Single-Storey Frame-Level Model



b) 3S Multi-Storey Frame-Level Model



c) 4S Multi-Storey Frame-Level Model

Figure 3.18. Scaled Ground Motions Records at Target Period

3.5.3 Results

Single Storey Frames

Figure 3.19 illustrates the drift responses obtained from the time history analysis of a one-storey frame subjected to the selected ground motion records. The figure also includes the average values for maximum and residual drift percentages. The results indicate that SMA connections exhibit larger peak displacements due to their slightly lower initial stiffness than steel connections, which offered a notable advantage in minimizing permanent deformations. The self-centering capability of SMA connections significantly enhances the seismic resilience of the structure, effectively reducing the need for post-earthquake repairs when buildings experience cyclic loading.

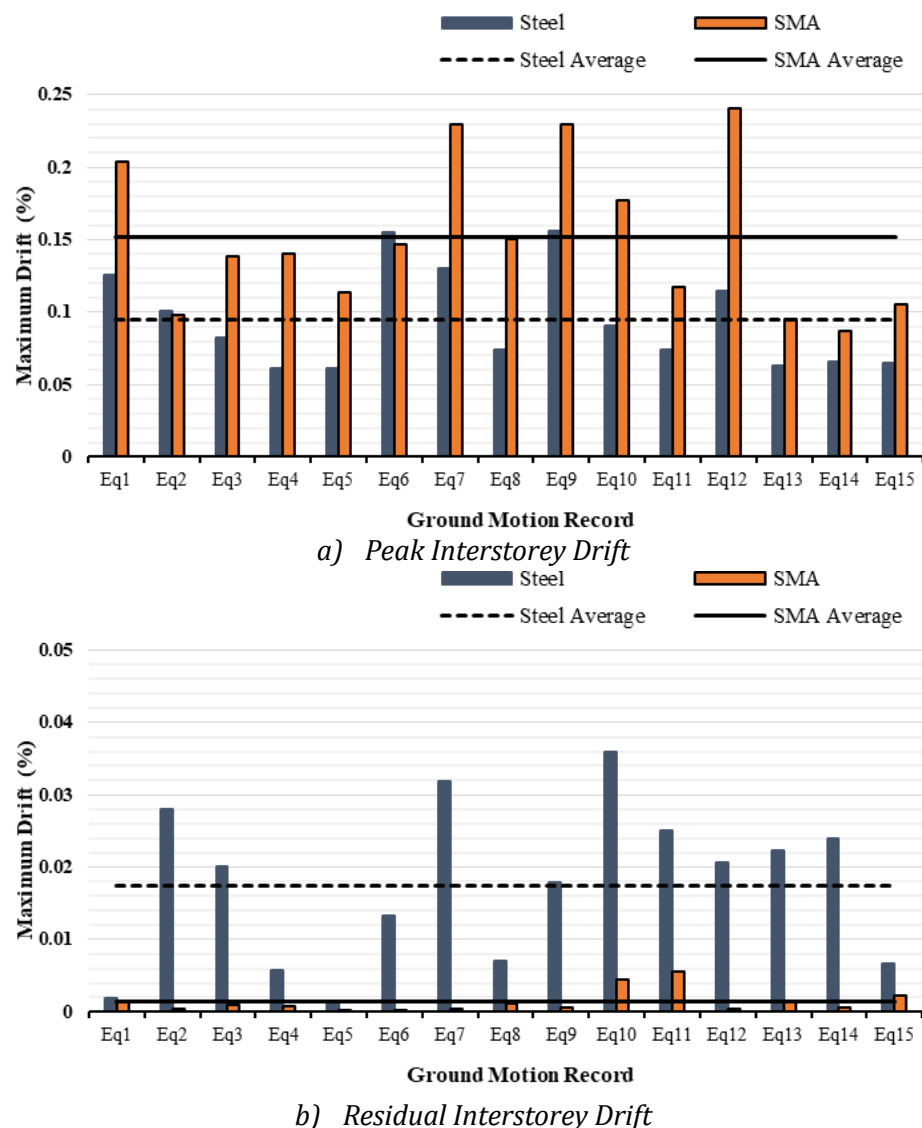


Figure 3.19. Single-Storey Frame – Interstorey Drifts

The numerical frames with SMA connections had higher peak displacements, with an average peak drift of 0.15%, compared to 0.095% for frames with steel connections. This means the frames with SMA connections experienced approximately 57% higher displacements than the frames with steel connections. The increased drift is attributed to the lower stiffness of SMA connections relative to steel ones, allowing more deformation under similar loading. Despite this, SMA frames exhibited superior self-centering behaviour. Residual drift for SMA frames averaged just 0.0014%, significantly lower than the 0.017% observed in steel frames, meaning the residual drift of the steel frames was on average over 12 times larger than that of the SMA frames. These results highlight the effectiveness of SMA connections in reducing residual deformations and post-earthquake damage, enhancing the seismic resilience of single-storey braced frames.

Multi-storey Frames

The results for the three-storey (3S) and four-storey (4S) braced timber frames are presented in Figure 3.20, detailing the peak drift, residual drift, and peak acceleration at each storey. The findings are consistent with those observed in the single-storey frame analysis, where SMA-connected frames exhibit higher maximum drift compared to steel-connected frames. In addition, the results for the multi-storey frames highlight differences in structural response attributed to the variation in stiffness between the two models. In the three-storey frame, the uneven stiffness between storeys creates variations in lateral resistance, leading to higher drift variability across the height. In contrast, the four-storey frame provides a more consistent stiffness, leading to a more uniform interstorey drift pattern. These differences highlight how bracing configuration impacts the dynamic response of the structures, with stiffness irregularities affecting lateral deformations, force redistribution, and overall seismic performance.

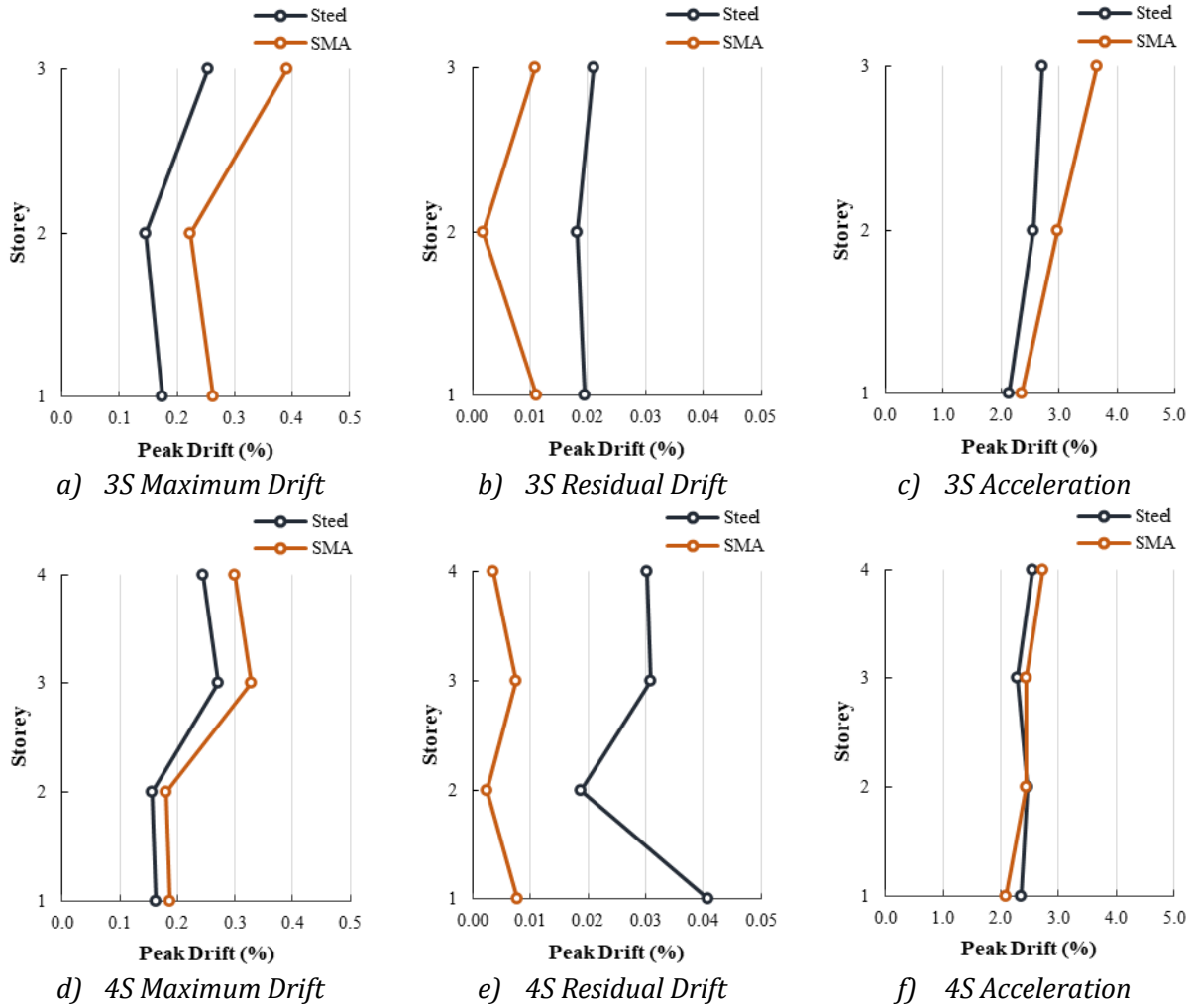


Figure 3.20. Multi-Storey Frame – Interstorey Drifts and Accelerations

In general, the frames with SMA connections generally exhibit larger peak drifts at all storeys compared to those with steel connections. In both the three- and four-storey cases, peak drift increases with height, a common trend under seismic loading. However, when examining residual drift, steel-connected frames demonstrate more pronounced permanent deformations, indicating that they do not fully recover after the ground motion stops. In contrast, SMA-connected frames maintain significantly lower residual drifts at every storey level, highlighting their self-centering capability and ability to regain their original position after seismic events. Both connection systems display similar trends regarding peak floor acceleration, with values increasing along the building height. While minor differences exist between the steel and SMA models, their acceleration profiles remain relatively similar. These results emphasize the advantage of minimizing permanent deformations in SMA-connection frames, demonstrating the seismic resilience of self-centering when using SMA dowels in multi-storey braced timber-frame connections.

CHAPTER 4. SEISMIC DESIGN OF BUILDING PROTOTYPES

4.1 Overview

The reliability of the modelling framework has been validated through the calibration of numerical spring models using experimental data for both steel- and SMA-bolted connections. This chapter now focuses on the seismic design of three- and six-storey mass timber prototype building representative of an Ottawa Class C site. Further information regarding the building selection was provided in Section 1.4. Both structural systems were designed following NBCC 2020 and CSA O86 provisions. The design incorporates experimental ductility values, categorizing the buildings as Seismic Category 3 (SC3) with limited ductility. This classification constrained the 12-meter prototype, while a 24-meter prototype intentionally exceeded the SC3 height limit to explore performance implications. Chapter 4 investigates the use of SMA bolts for connections in BTFs as an alternative for steel-bolted connections and provides the information required to build system-level numerical models to assess their seismic performance.

4.2 Building Details

The archetype building was composed of mass timber structure with a footprint of 42×42 metres, situated in Ottawa (City Hall). It was designed with a symmetrical configuration to ensure that the lateral force-resisting systems (LFRS) was identical in the X and Y directions. The structure was intended for office occupancy and was developed in two height variations, a three-storey (M3) and a six-storey (M6) configuration, with total heights of 12 metres and 24 metres, respectively. The design incorporated a consistent floor-to-floor height of 4 metres throughout the building.

The structural system consists of a post-and-beam timber structure as the gravity load-resisting system and uses glulam members with CLT slabs. The seismic force resisting system (SFRS) consists of twelve equally spaced braced glulam frames per direction (X and Y) in a symmetrical arrangement with four frames in each orientation. Each frame has a bay width of four metres, resulting in a width-to-height aspect ratio of 1.0. The typical floor plan, M3 building elevation, and M6 building elevation of the structure are illustrated in Figure 4.1, Figure 4.2, and Figure 4.3, respectively.

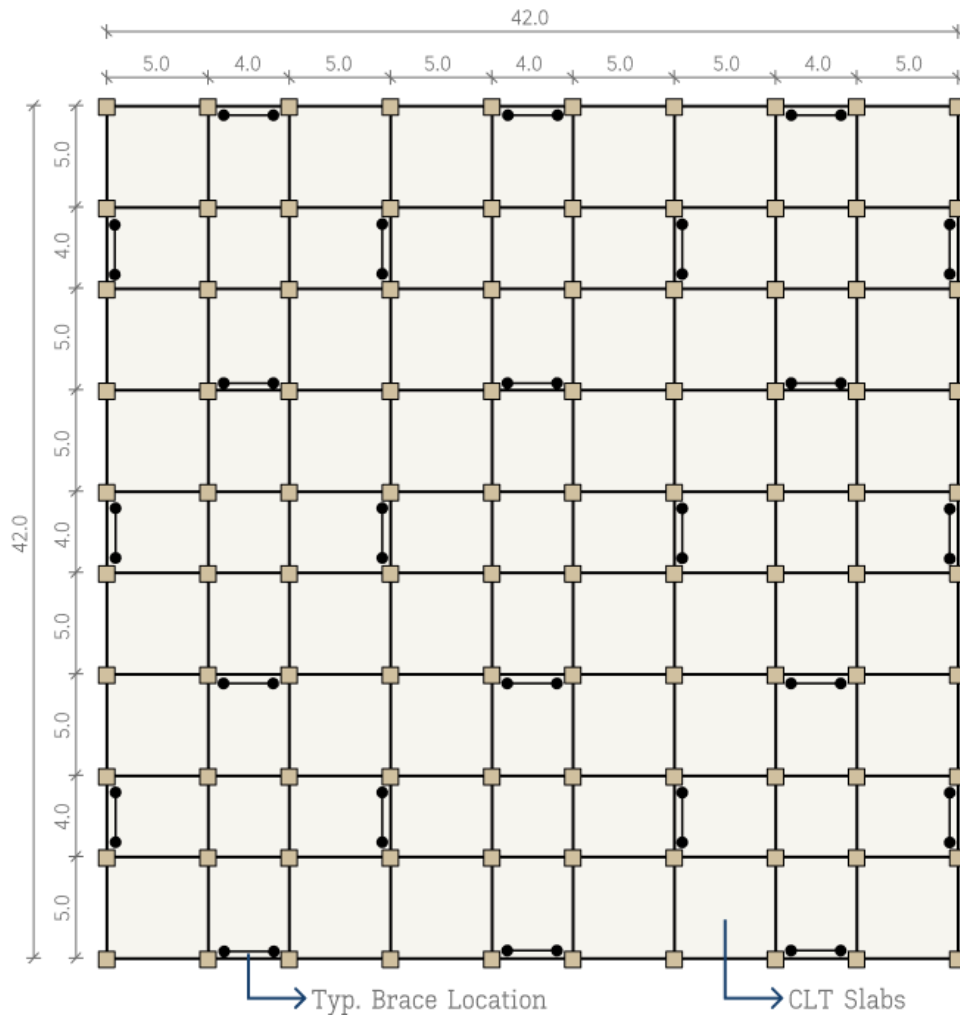


Figure 4.1. Prototype Building – Typical Storey Plan

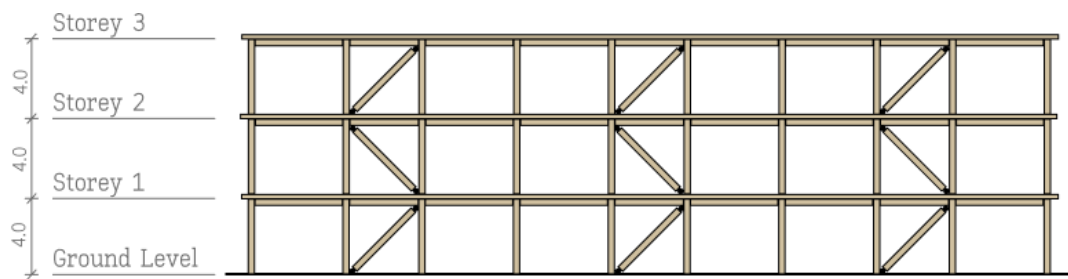


Figure 4.2. Prototype Building - Three-Storey (M3) Brace Timber Frame Elevation

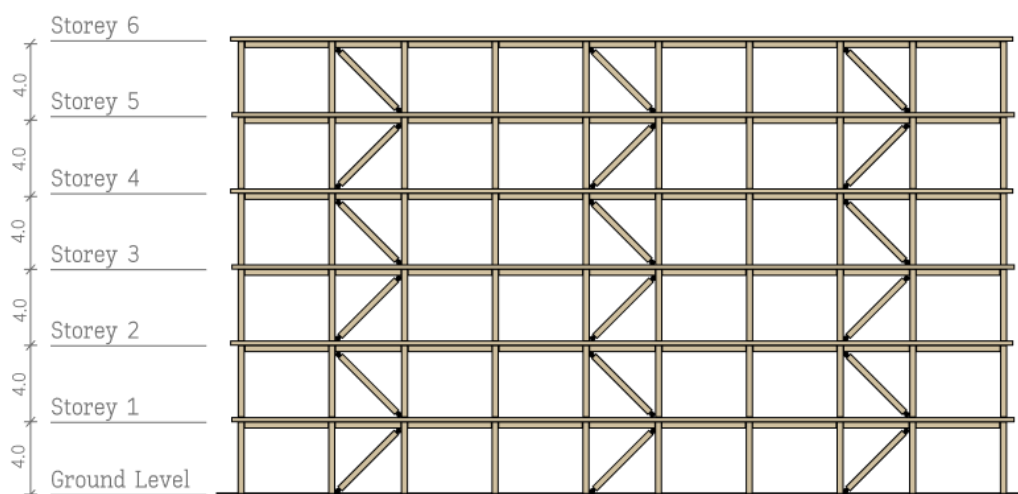


Figure 4.3. Prototype Building - Six-Storey (M6) Brace Timber Frame Elevation

The structural design assumed pinned connections between timber members, allowing the timber elements to remain elastic while ductility and energy dissipation are achieved through the brace connections. The glulam elements were specified as SPF sections in grades 12c-E and 20f-EX, while the CLT deck was detailed to be the E1 grade class. The mechanical properties of these members were determined following CSA O86 (2019) standards, and the structural analysis follows the provisions of the NBCC (2020).

4.3 Design Loads

4.3.1 Gravity Loads

The design loads were determined based on Section 4.1 from the NBCC (2020) for a building classified under the normal importance category ($I_E = 1.0$). Gravity loads, including the dead, live, and snow loads, are summarized in Table 4.1. The self-weight of the structure is not reported, as it is accounted for the static analysis model developed for the gravity and lateral-system analyses. The snow load at the roof level was established based on 1-in-50-year return period specific for the building location.

Table 4.1. Gravity Design Loads

Load	Type		Load (kPa)
Floor Levels			
	Self-weight	DL	-
	Dead Load	SDL	2.5
	Design Floor Live Load	LL	2.4
Roof Level			
	Self-weight	DL	-
	Dead Load	SDL	1.6
	Specified Snow Load	SL	2.3

4.3.2 Seismic Loads

The critical lateral loads consist of forces induced due to seismic activity, which will be determined based on the NBCC (2020) guidelines. To design the SRFs for the building archetypes, the Equivalent Static Force Method (ESFM) was used for a Site Class C in Ottawa (City Hall area). This site's characteristics have a peak ground acceleration (PGA) of 0.207 m/s^2 and peak ground velocity (PGV) of 0.147 m/s . The spectral response acceleration for a 2%-in-50-year return period in the selected site is shown in Figure 4.4.

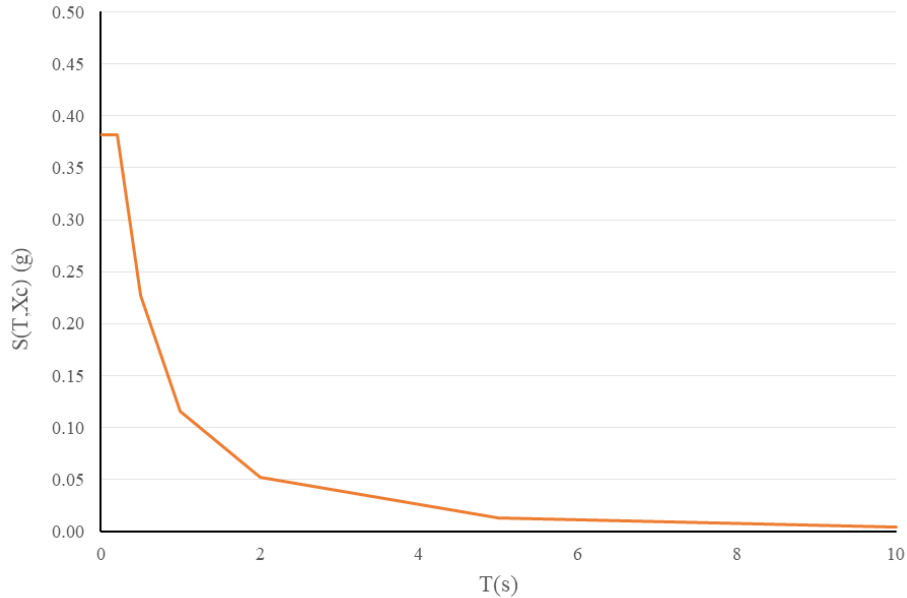


*Figure 4.4. Design Spectrum - Ottawa (City Hall), Site Class C
Retrieved from NBCC (2020)*

In accordance with the NBCC (2020), BTF buildings were assigned one of four seismic categories (SC) based on the more severe condition between the building importance and the spectral response indices, $I_{ES}(0.2)$ and $I_{ES}(1.0)$. These seismic categories not only set force modification factors (FMFs) but also impose building height restrictions that depend on the ductility of the braced frames. Table 4.2 presents the seismic categories alongside their corresponding height limitations and FMFs. BTFs with moderate ductility were limited to a maximum height of 20 metres, while those with limited ductility were restricted to 15 metres. The building was classified as SC3 since the $I_{ES}(0.2)$ value was 0.38 compared to an $I_{ES}(1.0)$ value of 0.12, making SC3 the governing condition.

Table 4.2. FMFs and Height Restrictions for BTFs
Adapted from NBCC (2020)

		BTF Ductility	
		Limited	Moderate
Rd		1.5	2.0
Ro		1.5	1.5

	I _{ES} (0.2)	I _{ES} (1.0)	Height Restrictions	
SC1	< 0.2	< 0.1	NL	NL
SC2	≥ 0.2 to < 0.35	≥ 0.1 to < 0.2	NL	NL
SC3	≥ 0.35 to ≤ 0.75	≥ 0.2 to ≤ 0.3	15	20
SC4	> 0.75	> 0.3	15	20

NL = no limit

To determine the ductility category of the BTF systems, the criteria defined by Chen and Popovski (2020) was used. The authors state that for achieving a moderate ductile BTF, the system requires a minimum connection ductility of 11.5 and 6.3 when yielding occurs at only one end or both ends of the brace, respectively (Chen & Popovski, 2020). For limited-ductility systems, the required connection ductility is 5.4 when only one end yields and 3.2 when yielding occurs at both ends (Chen & Popovski, 2020).

For the current study, the ductility of the connections was retrieved from the work by Cl  roux et al. (2024), which reports that the steel and SMA connections exhibited ductility values of 7.8 and 4.7, respectively. Although the steel connections satisfied the ductility requirements for moderately ductile frames, the SMA connections did not meet this requirement. Consequently, the BTF system used in the proposed prototype buildings was classified as a limited ductility, with assigned FMFs of $R_d=1.5$ and $R_o=1.5$.

As shown in Table 4.2, a BTF system with limited ductility, classified under SC3, is limited to a maximum height restriction of 15 metres. For this reason, the proposed archetype building was designed as a 12-meter structure (M3) that complies with the SC3 height restriction. The 24-meter structure (M6), however, exceeds the SC3 height limit, but was intentionally designed to assess the effects of exceeding this height restriction. The seismic weight, W , for each storey of both buildings was defined as the total of the dead loads and 25% of the snow loads. A summary of the details for the ESFM analysis for both archetype buildings is presented in Table 4.3, and additional details can be seen in Appendix B.

Table 4.3. ESFM Summary

	Prototype Building	
	M3	M6
Height, H [m]	12.0	24.0
No. Storeys	3	6
Specified weight, W [kN]	16459	34252
Empirical period, T_a [s]	0.3	0.6
Spectral acceleration, $S(T_a)$ [g]	0.33	0.205
Base shear, V [kN]	1863	3118

Due to the symmetrical floor plan and the even distribution of the SFRS, torsional effects were considered only for accidental eccentricity-induced torsional moments. The center of mass (CM) and center of rigidity (CR) were determined to coincide ($e_x=0$), eliminating inherent torsional effects. Furthermore, given the square footprint of the building, the accidental eccentricity was identical in both orthogonal directions. The procedure for determining accidental eccentricity was based on the torsional sensitivity factor (B_x), which was calculated at each level as the ratio of (1) the maximum storey displacement (d_{max}) caused by seismic forces applied at $\pm 10\%$ of the building width ($\pm 0.10D_{nx}$) from the CM, and (2) the average displacement (d_{avg}) when the seismic force was applied directly at the CM, both measured at the extreme points of each level. Since the torsional sensitivity (B_x) was below 1.7 across all levels for both archetype buildings, the accidental torsional effects were accounted for by applying torsional moments about the vertical axis. At each level, these moments were determined by incorporating eccentricity (e_x) using an additional amplification factor of $\pm 10\%$ of the building width, multiplied by the seismic force (F_x). These adjustments addressed uncertainties in mass distribution and structural stiffness.

To evaluate the influence of torsional moments on the braced frames, a three-dimensional SAP2000 model was developed. The slabs were modelled as rigid diaphragms, and the additional shear forces due to torsional moment at each frame were calculated. The maximum shear force at each frame was obtained by summing the direct seismic force (F_x) with the forces induced by torsional moments. The shear forces (V_{fx}) to perform the static analyses for the SFRS design are summarized in Table 4.4 and Table 4.5.

Table 4.4. Seismic Forces and Torsional Sensitivity for M3 Building

Level	F_x (kN)	V_x (kN)	d_{max} (mm)	d_{avg} (mm)	B_x	V_{fx} (kN)
R4	813.5	813.5	24.7	20.7	1.19	223.3
L3	699.7	1513.2	14.7	12.3	1.20	192.1
L2	349.8	1863.0	6.9	6.0	1.15	96.1
L1	-	1863.0	-	-	-	-

Table 4.5. Seismic Forces and Torsional Sensitivity for M6 Building

Level	F _x (kN)	V _x (kN)	d _{max} (mm)	d _{avg} (mm)	B _x	V _{fx} (kN)
R7	737.0	737.0	185.2	155.6	1.19	202.7
L6	792.3	1529.3	146.7	121.3	1.21	217.9
L5	633.9	2163.2	102.8	85.3	1.21	174.3
L4	475.4	2638.6	66.3	55.9	1.19	130.7
L3	316.9	2955.5	34.1	28.1	1.21	87.2
L2	158.5	3114.0	14.2	11.6	1.22	43.6
L1	-	3114.0	-	-	-	-

4.4 Structural Design

4.4.1 Gravity Structural System

The structural design of the timber elements was carried out per the CSA 086 (2019) standard. The section sizes were determined through an iterative design process to optimize structural efficiency while minimizing the number of unique sections to improve constructability. The member sizes, material grades, and mechanical properties were selected based on standard SPF glulam sections, as specified in CSA 086 (2019), for both strength and serviceability criteria, but fire rating was not considered in this design. The size of the columns at the braced frames was governed by the axial forces resulting from the lateral forces. A summary of the timber members for the proposed archetype buildings is provided in Table 4.6, and further details are provided in Appendix B.

Table 4.6. Design Summary of Columns and Beams

Member	Storeys	Specie	Grade	Size (mm)
M3 Building				
Column	1 - 3	SPF	12c-E	215x266
Beam	Floor	SPF	20f-EX	215x418
Beam	Roof	SPF	20f-EX	215x380
M6 Building				
Column	1 - 3	SPF	12c-E	315x380
Column	4 - 6	SPF	12c-E	215x228
Beam	Floor	SPF	20f-EX	215x418
Beam	Roof	SPF	20f-EX	215x380

4.4.2 Seismic Force Resisting System

Regarding the design of the SFRS, the BTF system featured an internal single knife plate connection with both steel and SMA bolts. This configuration mirrors the connection type used in the experimental study

conducted by Cl  roux et al. (2024). The design of the timber braces was directly influenced by the connection requirements; in particular, the glulam members were oversized to satisfy the structural criteria established for the connection design. This criterion approach was implemented due to the limited guidance in the current provisions in CSA O86 for detailing key aspects of brace connections to ensure the system-level ductility performance required for ductile failure. In this study, the fasteners were designed to yield prior to the initiation of brittle failure modes, allowing the connection to undergo significant plastic deformations and maintain a stable hysteretic response under cyclic loading.

The brace design process was intended to reduce the number of different brace and connection types across the structure, which is in line with common engineering design practices. The axial forces of the braces were governed by the load combination $1.0E + 1.0D + 0.5L + 0.25S$ (E: earthquake, D: dead, L:live, S: snow). The axial forces (in Kilonewtons) for the braces of both buildings are illustrated in Figure 4.5 and Figure 4.6. To minimize connection variation, each brace was scaled for the maximum tension force, F_t , experienced on each story of the M3 building and every other storey of the M6 building. Since the connection design governed the size of glulam members, compression forces were not critical.

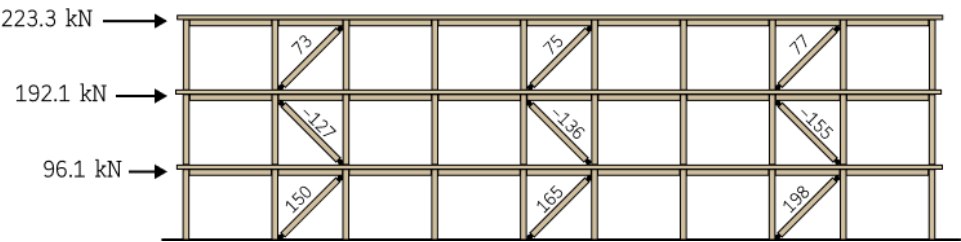


Figure 4.5. Brace Axial Forces – M3 Prototype Building

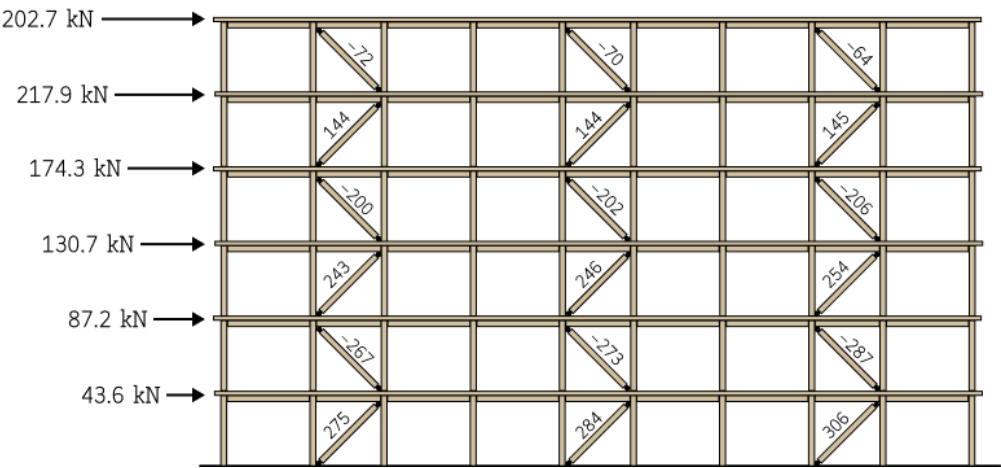


Figure 4.6. Brace Axial Forces – M6 Prototype Building

The axial forces illustrated in the figures above are presented as positive values when the brace members are in tension and negative when subjected to compression. It is crucial, however, to note that if the direction of the applied loads were reversed, the braces would exhibit an opposite axial behaviour while maintaining the same magnitude. Therefore, the governing design axial forces for each brace and connection were taken as the maximum forces identified at each storey. The connections designed for both prototype buildings (M3 and M6) used either steel or SMA bolts, yielding four distinct braced timber frame (BTF) systems: the M3 building with steel connections, the M3 with SMA connections, the M6 with steel connections, and the M6 with SMA connections. The brace and connection specifications were consistent across each storey to simplify the design process. However, in the case of the M6 prototype, brace and connection designs were consistent for pairs of consecutive storeys, resulting in three brace configurations per building prototype.

Experimental results presented by Cl  roux et al. (2024) demonstrated that SMA bolts exhibited slightly greater bending strength than conventional steel bolts. Consequently, SMA connections typically have the potential to achieve equivalent structural capacities using fewer fasteners than steel connections. However, this reduction in fastener quantity was not achieved, except in one instance where the SMA connections could have had a smaller number of fasteners than the steel connections, but it was decided to maintain the same number of fasteners for consistency. In the design of other structures, it is important to ensure comparable capacities between SMA and steel connections, as it facilitates a more meaningful evaluation of their relative seismic performance within BTF lateral systems. A detailed summary of brace and connection specifications, including the force resistance (F_r) values that governed the connection designs, is presented in Table 4.7.

The naming convention for the connections shown in Table 4.7 was developed to reflect the bolt material, bolt size, and connection configuration. The bolt material is denoted as SC for steel connections and NC for connections using SMA bolts. This is followed by the fastener diameter, d_F , expressed in imperial units -for example, 3/8-inch and 1/2-inch bolts are labelled as 3/8B and 1/2B, respectively. The connection configuration is further specified by the total number of fasteners, n_F , along with their layout described by the number of rows (n_R) and columns (n_C). For instance, the brace name SC-3/8B-20-4/5 refers to a steel connection using 3/8-inch (9.5 mm) bolts, with 20 fasteners arranged in 4 rows and 5 columns.

Table 4.7: BTF - Brace Design Summary for M3 and M6 Buildings

				Steel Connection Braces		SMA Connection Braces	
Storey	Force, F_t	Glulam		Name	Force, F_r	Name	Force, F_r
	(kN)	(mm)			(kN)		(kN)
M3 Building							
M3	1	198	265x342	SC-3/8B-20-4/5	219	NC-3/8B-20-4/5	248
M3	2	155	215x342	SC-3/8B-15-3/5	164	NC-3/8B-15-3/5	186
M3	3	77	175x304	SC-3/8B-9-3/3	99	NC-3/8B-9-3/3	112
M6 Building							
M6	1-2	306	265x418	SC-1/2B-16-4/4	308	NC-1/2B-16-4/4	349
M6	3-4	254	215x418	SC-1/2B-15-3/5	289	NC-1/2B-15-3/5	327
M6	5-6	145	175x418	SC-1/2B-9-3/3	173	NC-1/2B-9-3/3	198

These tables provide additional details about the brace design, including the connection configurations and corresponding design criteria. As previously stated, the seismic performance of the braces is closely linked to achieving ductile failure in the connections, which is critical for improving energy dissipation during seismic events. The ductile failure in connections includes modes (d), (e), and (g) based on CSA O86 equations (2019). These failure modes account for a combination of wood embedment strength and fastener yielding. Among these, mode (g) failure, which involves the formation of plastic hinges at the wood-steel shear plane, was particularly desirable as it enhances ductility and energy dissipation.

To achieve mode (g) failure, two ductility factors were incorporated into the connection design, in accordance with bolt-type connection specifications from CSA O86.

a) Slenderness Ratio

The first factor influencing ductile failure was the slenderness ratio, defined as the wood thickness to bolt diameter ratio. Several research studies discussed in the literature reported that higher slenderness ratios promote ductile failure in dowel-type connections. Experimental testing by Baird et al. (2023) confirmed that a slenderness ratio of 12 facilitates significant plastic deformation, improving the overall energy dissipation capacity of the connection. To incorporate this finding into the design process, a slenderness ratio check (SR) was introduced, ensuring compliance with the recommended slenderness criteria.

b) Ductile-to-Brittle Failure Ratio

The second factor was the ductile-to-brittle failure ratio (DBR), which ensured that yielding failure modes dominated while preventing brittle failure mechanisms such as row shear, group tear-out, or tension failure. This ratio was determined by comparing the yielding resistance with the minimum brittle failure resistance, following the CSA O86 standards. The DBR was limited to 2.0, indicating that the connection required twice the force to fail in a brittle manner compared to a ductile failure mode.

The seismic resilience of the BTF system was enhanced by implementing ductility-based design criteria, ensuring controlled yielding behaviour and increased energy dissipation. The connection design was governed by the $DBR \geq 2.0$ criteria, requiring the use of small-diameter bolts 9.5 mm (3/8 inches) and 12.7 mm (1/2 inches) for M3 and M6 building connections, respectively. The small bolt diameters, relative to the wood thickness, resulted in high slenderness ratios ($SR \geq 16.0$), further promoting ductile failure modes. Additionally, to meet the required resistance, a larger number of fasteners was required. While using larger-diameter bolts with fewer fasteners could have provided equivalent resistance, this approach would have failed to satisfy the DBR and SR requirements.

Table 4.8. M3 Building - Brace Design & Ductility Factors Summary

Brace Name	Bolt Material	No. Storeys	Specie	Grade	Glulam Size (mm)	d _F (mm)	n _R x n _C	S _p / S _q	SR	BDR
SC-3/8B-20-4/5	Steel	1	SPF	20f-EX	265x342	9.5	4 x 5	65 / 65	18	2.2
SC-3/8B-15-3/5	Steel	2	SPF	20f-EX	215x342	9.5	3 x 5	65 / 65	18	2.2
SC-3/8B-9-3/3	Steel	3	SPF	20f-EX	175x342	9.5	3 x 3	70 / 50	18	2.1
NC-3/8B-20-4/5	SMA	1	SPF	20f-EX	265x342	9.5	4 x 5	65 / 65	18	2.0
NC-3/8B-15-3/5	SMA	2	SPF	20f-EX	215x342	9.5	3 x 5	65 / 65	18	2.0
NC-3/8B-9-3/3	SMA	3	SPF	20f-EX	175x342	9.5	3 x 3	70 / 50	18	2.1

Table 4.9. M6 Building - Brace Design & Ductility Factors Summary

Brace Name	Bolt Material	No. Storeys	Specie	Grade	Glulam Size (mm)	d _F (mm)	n _R x n _C	S _p / S _q	SR	BDR
SC-1/2B-16-4/4	Steel	1 - 2	SPF	20f-EX	265x418	12.7	4 x 4	100/100	16	2.3
SC-1/2B-15-3/5	Steel	3 - 4	SPF	20f-EX	215x418	12.7	3 x 5	100/100	16	2.3
SC-1/2B-9-3/3	Steel	5 - 6	SPF	20f-EX	175x418	12.7	3 x 3	100/100	16	2.3
NC-1/2B-16-4/4	SMA	1 - 2	SPF	20f-EX	265x418	12.7	4 x 4	100/100	16	2.0
NC-1/2B-15-3/5	SMA	3 - 4	SPF	20f-EX	215x418	12.7	3 x 5	100/100	16	2.0
NC-1/2B-9-3/3	SMA	5 - 6	SPF	20f-EX	175x418	12.7	3 x 3	100/100	16	2.0

It is important to note that ductile failure in the connection is determined by computing the unit lateral yield resistance, which is a function of the bending performance of the fasteners. This resistance is calculated using both the yield strength (f_y) and the ultimate strength (f_u) of the fastener in bending. The connection design in this investigation was determined using the bending strength of steel and SMA dowels reported in the work by Cl  roux et al (2024). In their material characterization program, Cl  roux et al. (2024) assessed 6.35 mm diameter fasteners using the Standard Test Method for Determining the Bending Yield Moment of Nails (ASTM, 2021). The yield strength was established using the 5% offset method (ASTM, 2021), while the ultimate strength was defined as the maximum load reached during testing. According to their findings, the yield strength of the steel and SMA dowels was 439 MPa and 571 MPa, respectively, while the ultimate strength was reported as 537 MPa for steel fasteners and 1082 MPa for SMA fasteners (Cl  roux et al., 2024). These bending properties informed the basis for calculating the lateral resistance of the connections and ensuring ductile failure behaviour.

4.5 Summary

This chapter focuses on the structural design of the two prototype building of an Ottawa Class C site following NBCC and CSA O86 provisions. The design includes the use of SMA bolts for connections in BTFs as an alternative to steel-bolted connections, providing detailed information to build system-level numerical models to assess their seismic performance. There are four BTF systems designed, two three-storey frames and two six-storey frames, each one uses either steel or SMA bolted connections. The aim of this chapter is to provide with the foundation for subsequent chapters by establishing the design parameters and configurations needed for numerical modelling and seismic performance assessment.

CHAPTER 5. NUMERICAL MODELLING AND ANALYSIS

5.1 Overview

Chapter 5 details the numerical modelling and implementation of nonlinear time history analyses in OpenSeesPy to assess the seismic response of the BTFs developed in Chapter 4 under existing earthquake records. This chapter begins with developing uniaxial material models in OpenSeesPy to represent connection springs using beam-on-elastic-foundation stiffness estimates, ductility-based displacement limits, and energy-consistent degradation rules. These spring connections were used to develop BTF in OpenSeesPy that represent the interior frames of the designed prototype buildings. A four-step analysis framework was implemented that consists of a gravity routine, modal analysis, Rayleigh damping, and nonlinear time history analysis. This framework assesses the structural response of fifteen scaled ground motion records following NBCC 2020 guidelines. The numerical modelling and analysis developed in this chapter conveys the framework required to assess code compliance BTF buildings, specifically the difference in the structural response of SMA and steel dowel-type connections.

5.2 Modelling of Connections

To simulate the hysteretic response of bolt-type connections used in the BTF archetype structures, mechanical properties were defined and implemented in OpenSees through nonlinear spring models. These connection parameters, including yield and peak strength, initial stiffness, yield and ultimate displacements, and other nonlinear mechanical behaviour, were derived through a structured calibration process grounded in experimental data and supported by theoretical models. To determine the yield and peak strengths of the numerical brace connections in this study, benchmark cyclic test data from Cl  roux et al. (2024) were scaled using capacity-based design parameters.

In addition, the connection stiffness has been determined using the simplified beam-on-elastic-foundation theory (BEFT) developed by Wang et al. (2021). They also described an offset approach to calculating yield displacement, which was adapted to estimate the displacement of the connections. Finally, three calibration methods were used to obtain the additional nonlinear parameters required to define the DTE material models, including stiffness degradation ratios, pinching effects, unloading and reloading stiffness, and energy-based damage factors. These calibration procedures provided an appropriate foundation for establishing parameters, resulting in the proper representation of both the elastic and inelastic behaviours.

5.2.1 Calibration Definition

Yield and Peak Strength

The cyclic test results from the reference experiment conducted by Cl  roux et al. (2024) were used as benchmark input to develop a scaling framework for generating the hysteresis loops of the spring models in the M3 and M6 system-level models. A similar methodology was previously introduced by Chen and Popovski (2020, 2021) in their studies on BTFs with riveted connections and the relationship between BTF system behaviour and connection ductility. In both studies, the hysteresis loops of riveted connections from glulam brace specimens, tested by Popovski (2004), were utilized as benchmark inputs to develop equivalent nonlinear connector elements, followed by analytical pushover analyses. In the BTF system and connection ductility study by Chen and Popovski (2020), ductility estimates obtained from numerical models were compared with those derived from ductility relationship equations, revealing a 5% difference between the two methods. These findings demonstrate that using reference experiments to develop numerical spring models for connections with larger capacity is a practical approach, as it has been effectively implemented in prior studies.

In the presented study, the hysteresis loops of the diagonal brace connections were derived by scaling the force of the benchmark hysteresis loops using a design factor. The resistance of the benchmark connections was established as the average of the tension and compression hysteresis loops from the cyclic connection tests conducted by Cl  roux et al (2024). The capacity factor was defined as the ratio of the design resistance, $F_{r,CSA}$, calculated using CSA O86 equations, to the testing resistance of the benchmark connections, $F_{r,test}$. In addition, the maximum force factor was determined from the benchmark hysteresis loops, representing the ratio of peak force to yield force. The factors used to calculate both the yield force ($F_{r,factor}$) and maximum force ($F_{max,factor}$) are summarized in Table 5.1. These factors determined the yield, F_y , and peak, F_{max} , strength for the numerical connections based on the capacity resistance, F_r , defined during the design of the connections. These strength values are shown in Table 5.2 and Table 5.3.

Table 5.1: Yield and Maximum Force Factors

Connection	$F_{r,test}$	$F_{r,CSA}$	$F_{r,factor}$	$F_{max,factor}$
	kN	kN		
4SC _{avg}	18.2	16.4	1.11	1.75
4NC _{avg}	24.9	20.9	1.19	1.69

Table 5.2: M3 Building - Yield and Peak Force of Connections

Connection	Bolt Material	F_r	F_y	F_{max}
		kN	kN	kN
SC-3/8B-20-4/5	Steel	219	243	425
SC-3/8B-15-3/5	Steel	164	182	319
SC-3/8B-9-3/3	Steel	99	110	192
NC-3/8B-20-4/5	SMA	248	295	499
NC-3/8B-15-4/5	SMA	186	221	374
NC-3/8B-9-3/3	SMA	112	133	225

Table 5.3: M6 Building - Yield and Peak Force of Connections

Connection	Bolt Material	F_r , CSA	F_y	F_{max}
		kN	kN	kN
SC-1/2B-16-4/4	Steel	308	342	598
SC-1/2B-15-3/5	Steel	289	321	561
SC-1/2B-9-3/3	Steel	173	192	336
NC-1/2B-16-4/4	SMA	349	415	702
NC-1/2B-15-3/5	SMA	327	389	658
NC-1/2B-9-3/3	SMA	198	236	398

Connection Stiffness

The lateral load capacity of a bolt in a connection can be theoretically determined using the beam-on-elastic-foundation theory, a concept initially introduced by Winkler (1867). Kuenzi (1955) later applied the BEFT model to describe the interaction between the bolt and the surrounding wood, which behaves like a beam resting on an elastic foundation. In this method, the behaviour of a bolt within a single shear connection is described using a differential equation that governs the deflection curve of a beam supported by an elastic foundation, as illustrated in Figure 5.1. The differential equation (5.1) was introduced by Kuenzi (1955) and assumes that the reaction force of the foundation is proportional to the deflection of the beam, while neglecting shear deformations within the beam. In this equation, EI is the bending stiffness, y is the deflection at point x , and k is the foundation modulus.

$$EI \frac{d^4 y}{dx^4} = -ky \quad (5.1)$$

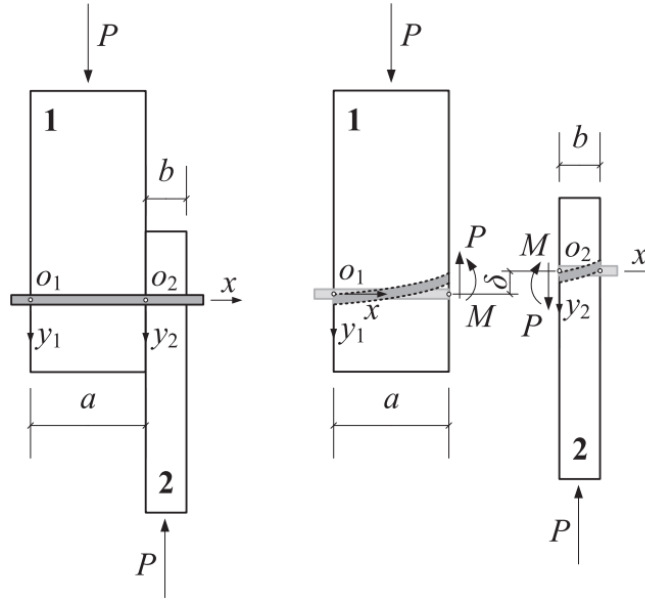


*Figure 5.1: Single Shear Connection Diagram
Retrieved from Wang et al. (2021)*

Recently, Wang et al. (2021) introduced a method for calculating the stiffness of bolted connections by adapting the BEFT to reflect the deformation mechanisms of such connections. This modified approach, referred to as the simplified BEFT (BEFT,s), incorporates reasonable assumptions that make it applicable to bolted joints. The deformation behaviour used in BEFT,s is derived from the European Yield Model (EYM) (Johansen, 1949), which is also implemented in the CSA 086 standard (2019) for determining connection strength. BEFT,s estimates connection stiffness in the pre-yield range and categorizes the deformation response into three distinct modes: Pre-Yield Modes I, II, and III. This study focuses specifically on Pre-Yield Mode III, which is of particular importance as it includes connections anticipated to yield in European Yield Model (EYM) Modes IIIm, IIIs, and IV, corresponding to modes (d), (e), and (g) in CSA 086 equations (2019). Additionally, Wang et al. (2021) emphasized that Pre-Yield Mode III is characteristic of connections with high slenderness ratios, which aligns with the slenderness parameters of the specimens in this study ($SR \geq 12$). The expression for calculating the stiffness associated with Pre-Yield Mode III ($K_{e,s,III}$) for both single and double shear configurations developed by Wang et al. (2021) is presented in equation (5.2).

$$K_{e,s,III} = \frac{4EI \lambda_1^3 \lambda_2^3}{(\lambda_1 + \lambda_2)(\lambda_1^2 + \lambda_2^2)} \quad (5.2)$$

This equation is governed by the characteristic parameter, λ_i , which quantifies the stiffness ratio between the bolt and the surrounding wood foundation. This parameter is derived from the BEFT differential

equation, reflecting the interaction between the bolt and the elastic medium. The characteristic parameter derived by Kuenzi (1955) is presented in equation (5.3).

$$\lambda_i = \sqrt[4]{\frac{k_i}{4EI}} \quad (i = 1, 2) \quad (5.3)$$

Wang et al (2021) also proposed empirical equations for determining the foundation modulus, k , in both directions, parallel- and perpendicular-to-grain. These equations were developed based on experimental testing results for bearing stiffness by Wang et al (2021). The loading equation for parallel-to-grain, shown in Equation (5.4), is applicable to this study since it corresponds to the uniaxial loading orientation of the braced timber connections. This expression relates the parallel-to-grain dowel-bearing stiffness, k_{II} , the bolt diameter, d , and oven-dry specific gravity, S_d .

$$k_{II} = 141.46 S_d^{1.35} d^{0.83} \quad (5.4)$$

The specific gravity for the glulam, $S_{d, \text{wood}}$, was determined as 0.44 based on the characterization of the materials results presented by Cl  roux et al. (2024). In regards to the 300W structural steel, the density was retrieved as 7850 kg/m³ (specific density, $S_{d, \text{steel}}$, is equal to 7.85) based on the CSA G40.21 (2013) standard, which governs the fabrication of welded structural quality steels such as 300W. Using these material properties, the foundation modulus for each medium was calculated. For 9.5 mm bolts, the modulus was calculated as 303 kN/mm in the wood substrate and 14,799 kN/mm in the steel, and for 12.7 mm bolts, the values increased to 385 kN/mm and 18,831 kN/mm for wood and steel, respectively. These foundation modulus values were then applied in equations (5.2) and (5.3) to compute the connection stiffness, K_e , for the brace connections in the BTF building. The resulting stiffness values are presented in Table 5.4 and Table 5.5.

Table 5.4: M3 Building - Brace Connection Stiffness Calculation Summary

Connection	Bolt Material	d_F mm	λ_1	λ_2	$K_{e, \text{single}}$ kN/mm	K_e kN/mm
SC-3/8B-20-4/5	Steel	9.5	0.03119	0.08247	6.16	123
SC-3/8B-15-3/5	Steel	9.5	0.03119	0.08247	6.16	92
SC-3/8B-9-3/3	Steel	9.5	0.03119	0.08247	6.16	55
NC-3/8B-20-4/5	SMA	9.5	0.03295	0.08524	4.13	83
NC-3/8B-15-4/5	SMA	9.5	0.03295	0.08524	4.13	62
NC-3/8B-9-3/3	SMA	9.5	0.03295	0.08524	4.13	37

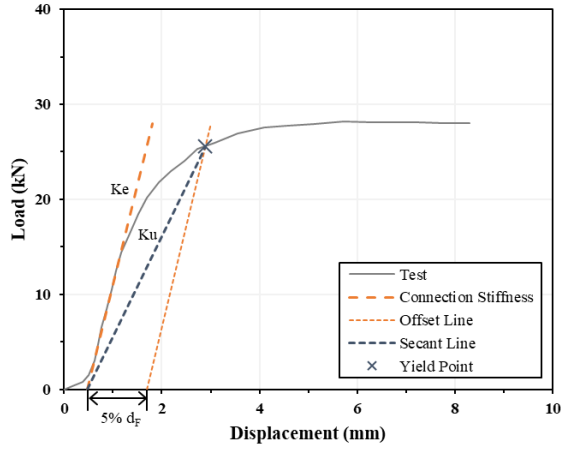
Table 5.5: M6 Building - Brace Connection Stiffness Calculation Summary

Connection	Bolt Material	d_F	λ_1	λ_2	$K_{e, \text{single}}$	K_e
		mm			kN/mm	kN/mm
SC-1/2B-32-4/8	Steel	12.7	0.02478	0.06552	9.86	158
SC-1/2B-24-4/6	Steel	12.7	0.02478	0.06552	9.86	148
SC-1/2B-15-3/5	Steel	12.7	0.02478	0.06552	9.86	89
NC-1/2B-28-4/7	SMA	12.7	0.02618	0.06772	6.61	106
NC-1/2B-20-4/5	SMA	12.7	0.02618	0.06772	6.61	99
NC-1/2B-12-3/4	SMA	12.7	0.02618	0.06772	6.61	60

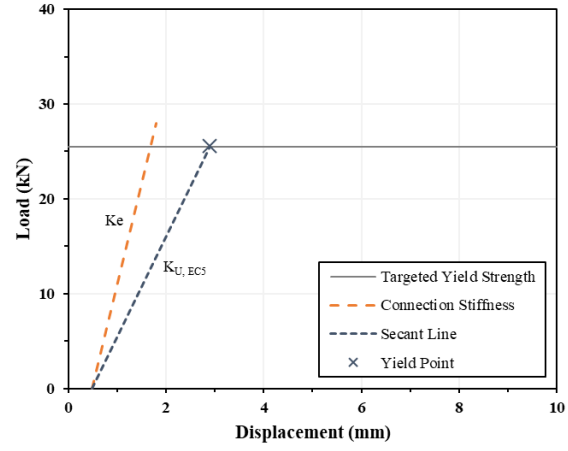
Yield and Ultimate Displacement

Wang et al (2021) also presented a method for determining the yield point based on the experimental curves used in their study, as illustrated in Figure 5.2a. The slope in the initial linear portion of the load-displacement curve was defined as the connection stiffness, K_e . The yield point is determined using an offset method, which involves shifting the initial stiffness line horizontally by 5% of the dowel diameter ($0.05d$). The yield point is defined as the intersection of this offset line and the load-displacement curve (X. T. Wang et al., 2021). The secant stiffness, K_u , at this yield point shows the slope of a line connecting the origin and the yield point. Based on test results, Wang et al (2021) reported a ratio K_e/K_u of 0.701, 0.683, and 0.673 for bolt diameters of 12 mm, 16 mm, and 18 mm, respectively. These findings are consistent with the Eurocode 5 standard, which states that the stiffness at the ultimate limit state should be approximated as $2/3$ of the serviceability limit state (CEN, 2004).

In this study, the secant stiffness, K_u , could not be determined directly since there were no experimentally recorded load-displacement curves. Instead, a simplified method was used, based on the Eurocode 5 guidelines and compatible with the findings of Wang et al. (2021), the secant stiffness, $K_{u, EC5}$, was assumed to equal $2/3$ of connection stiffness, K_e . After the secant stiffness, $K_{u, EC5}$, has been defined, the yield displacement, D_y , was calculated by taking the intersection of the secant stiffness line and the yield strength. Figure 5.2a shows the method used to determine connection stiffness from the load-displacement curve proposed by Wang et al (2021), whereas Figure 5.2b depicts the strategy utilized in this study to determine yield displacement.



a) Connection Stiffness Definition.
Adapted from Wang et al. (2021)



b) Yield Displacement Definition

Figure 5.2: Yield Point Definition

The ultimate displacement was determined based on the ductility ratios reported from the benchmark connection. Cl  roux et al (2024) reported ductility ratios for single and group configurations of steel and SMA connections under monotonic and cyclic loading conditions. The ductility ratios observed under cyclic loading in group connections (7.8 for steel bolt fasteners and 4.7 for SMA bolt fasteners) were used as the basis for calculating the ultimate displacement in the present study.

Other Material Model Parameters

To define the numerical load-displacement curves using the DTE material model, the remaining connection parameters were selected by considering the specific hardening, unloading, reloading, and pinching behaviour prescribed by each model based on the average calibrated models for steel and SMA connections. Many of these parameters are inherently linked to the initial stiffness of the connection. As a result, the average parameter values obtained during the calibration phase in the development of connection-level model calibration were used directly, a method known as direct stiffness calibration. These parameters are considered material-dependent in the calibration process from experimental testing.

In contrast, certain force- and stiffness-related characteristics are independent of other calibration values. For these, a simplified derivation approach was used, known as derived calibration, which is based on stiffness ratios determined via direct stiffness calibration. In addition, the parameters categorized as factors which are related to the stiffness degradation, curvature, damage, and amplification of the hysteresis curves were directly obtained from the average calibrated models since they rely on unitless values that characterize connection behaviour independently of other model parameters. This approach is known as

direct factor calibration. A complete overview of the calibration methodology applied to the material model definition is summarized in Table 5.6.

Table 5.6: Additional Material Model Parameters Method Definition

DTE Material Model Parameters		
Parameter	Description	Method
R_U	unloading stiffness ratio	direct stiffness calibration
c	hysteresis curvature factor	direct factor calibration
β	amplification factor for $D_{\max}$	direct factor calibration
γ	energy based damage factor	direct factor calibration
η	force intercept factor	direct factor calibration
α_p	stiffness degradation factor for pinching line	direct factor calibration
α_u	stiffness degradation factor for unloading line	direct factor calibration
α_r	stiffness degradation factor for reloading line	direct factor calibration
R_1	stiffness ratio for hardening line	direct stiffness calibration
F_i	force intercept	derived calibration
K_p	pinching line stiffness	derived calibration
K_d	descending line stiffness	derived calibration

5.2.2 Material Models

The connection-level models introduced in Chapter 3 were carried over to define the connection for the M3 and M6 system-level models. In particular, the numerical framework employs ZeroLength spring elements in OpenSees using the DTE material described previously. These spring hinge models were determined using the previously reported calibration procedure: (1) Scale the design capacity values using the strength factors, (2) utilize the BEFTs equations to determine elastic stiffness, (3) specify the yield and ultimate displacements using the 5% fastener diameter offset method and experimental ductility ratios, and (4) obtain additional parameters from the average calibrated models.

Table 5.7 shows the material-dependent constants that represent stiffness degradation, pinching, and unloading during connection-level calibration. These constants are not affected by connection capacity. The parameters ensure that the numerical representation is consistent with the experimental behaviour identified during the original test-based model development.

Table 5.7: Constant Values - DTE Model Parameters

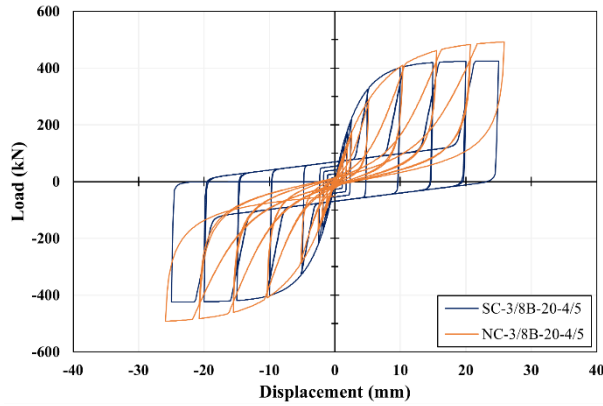
Parameters	Steel	SMA
R_U	10.0	10.0
c	0.2	1.1
β	1.02	1.02
γ	1.05	1.05
η	0.00075	0.0075
α_p	0.02	0.65
α_u	0.05	0.05
α_r	0.05	0.35
R_1	0	0

M3 Building

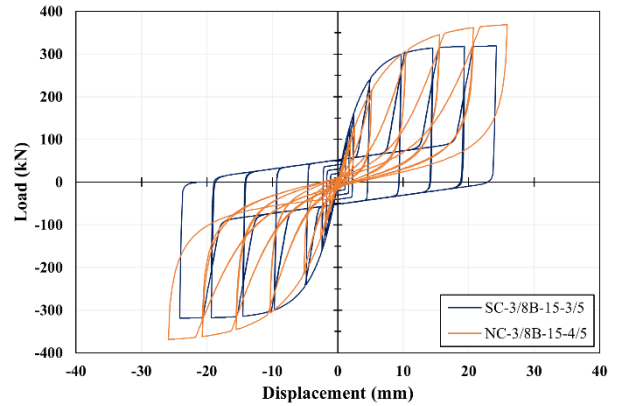
Table 5.8 presents the calibrated parameters for the DTE material models used for the M3 building prototype connections. The strength and stiffness values for the DTE models, including F_i , K_p , K_0 , F_0 , and K_d , were determined using the connection capacity and stiffness results from Table 5.2 and Table 5.4. The displacement-related inputs for both models were determined using the 0.05d offset approach for yield displacement, D_y , then scaled based on the experimental ductility ratios to define ultimate displacement limits, D_c . The material-dependent constants for the DTE models were obtained from Table 5.7. These parameters were used to simulate the hysteretic response of the M3 BTF connections under the cyclic loading protocol defined in Table 3.10. Figure 5.3 shows the resulting numerical hysteresis curves.

Table 5.8: M3 Building - DTE Model Parameters

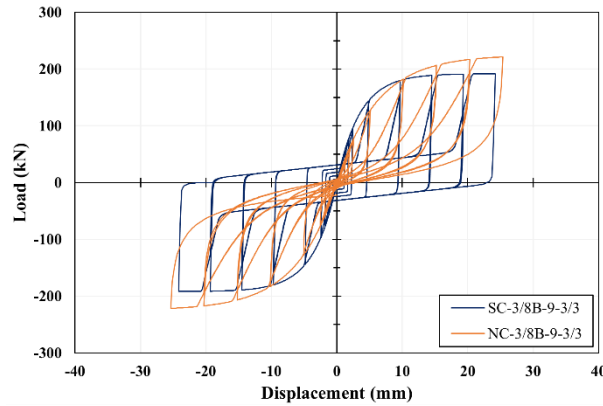
Connection	F_i	K_p	D_y	K_0	F_0	D_c	K_d
	kN		mm	kN/mm	kN	mm	kN/mm
SC-3/8B-20-4/5	69.3	3.0	3.2	123	425	25.0	776
SC-3/8B-15-3/5	51.9	2.2	3.1	92	319	24.2	582
SC-3/8B-9-3/3	31.3	1.3	3.1	55	192	24.2	349
NC-3/8B-20-4/5	12.0	11.9	5.5	83	499	25.9	388
NC-3/8B-15-3/5	9.0	8.9	5.5	62	374	25.9	291
NC-3/8B-9-3/3	5.4	5.3	5.4	37	225	25.4	175



a) Storey 1 – Connections Behaviour



b) Storey 2 – Connections Behaviour



c) Storey 3 – Connections Behaviour

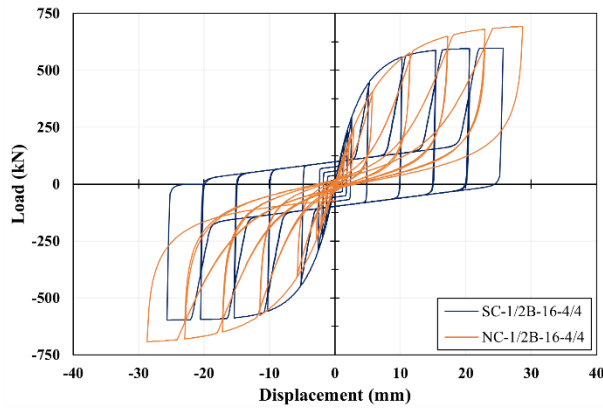
Figure 5.3: M3 Building - Connections Hysteresis Curves

M6 Building

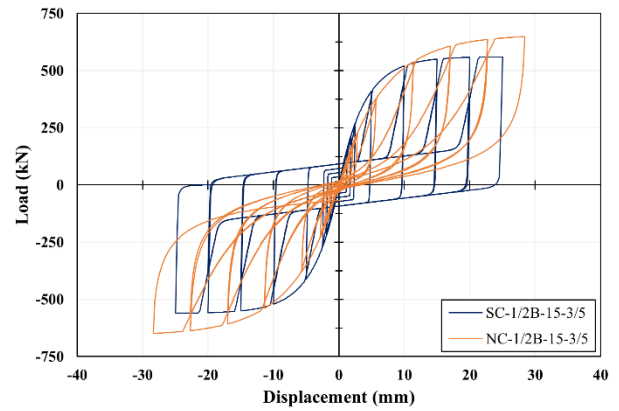
The spring models developed for the six-storey building prototypes were established using the same calibration procedure applied to the M3 connections. While the methodology remained consistent, the resulting parameters reflect the increased force and displacement demands associated with the M6 configuration, yielding a similar qualitative behaviour to M3 but at higher magnitudes. The connection capacity and stiffness values were drawn from Table 5.3 and Table 5.5, while the material-dependent constants were adopted directly from Table 5.7. The complete set of calibrated parameters is summarized in Table 5.9. These parameters were used to generate the numerical hysteresis curves for the M6 brace connections under the same cyclic loading protocol, and the results are presented in Figure 5.4.

Table 5.9: M6 Building - DTE Model Parameters

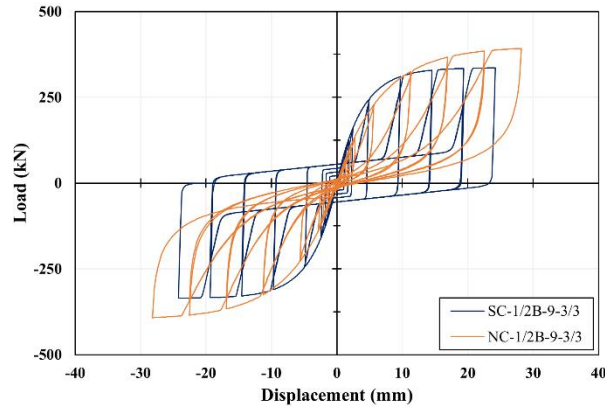
Connection	F_i	K_p	D_y	K_0	F_0	D_c	K_d
	kN		mm	kN/mm	kN	mm	kN/mm
SC-1/2B-16-4/4	98	3.8	3.3	158	598	25.7	994
SC-1/2B-15-3/5	92	3.6	3.2	148	561	25.0	932
SC-1/2B-9-3/3	55	2.1	3.1	89	336	24.2	559
NC-1/2B-16-4/4	17	15.2	6.1	106	702	28.7	497
NC-1/2B-15-3/5	16	14.3	6.1	99	658	28.4	466
NC-1/2B-9-3/3	10	8.6	6.0	60	398	28.2	280



a) Storeys 1 and 2 – Connections Behaviour



b) Storeys 3 and 4 – Connections Behaviour



c) Storeys 5 and 6 – Connections Behaviour

Figure 5.4: M6 Building - Connections Hysteresis Curves

5.3 Modelling of Building Prototypes

To evaluate the effect of SMA dowel-type connections on the seismic behaviour of BTF systems compared to traditional steel connections, numerical models were developed in OpenSees using its Python interpreter. These system-level models represent the interior frames of two archetype office buildings introduced in Chapter 4: a three-storey (M3) and a six-storey (M6) BTF structure. Each building was analyzed with steel and SMA connections at the ends of the timber braces. All models incorporate the DTE spring models calibrated previously in Section 5.2.2. A total of four building system-level models were created for this study, which are defined in Table 5.10.

Table 5.10: Numerical Models Developed

Building Prototype	Steel	SMA
M3 Building	THA-M3-SC	THA-M3-NC
M6 Building	THA-M6-SC	THA-M6-NC

The system-level frames were created as two-dimensional models, each node having three degrees of freedom. Glulam elements (columns, beams, and diagonal braces) were modelled as elastic components using elasticBeamColumn elements with properties based on the design details described in Chapter 4. The P-delta transformation for geometric stiffness was applied to account for P-delta effects in the analyses. All nonlinear behaviours occur at brace connections through zeroLength elements. Connections of each diagonal brace employed the concept of stronger and weaker connections described in Chapter 3. This ensured that nonlinear ductility and energy dissipation are only governed by connection properties, aligning with the capacity-based approach adopted for the design of the buildings. Representative diagrams of the timber frames modelled in OpenSees are provided in Figure 5.5 and Figure 5.6.

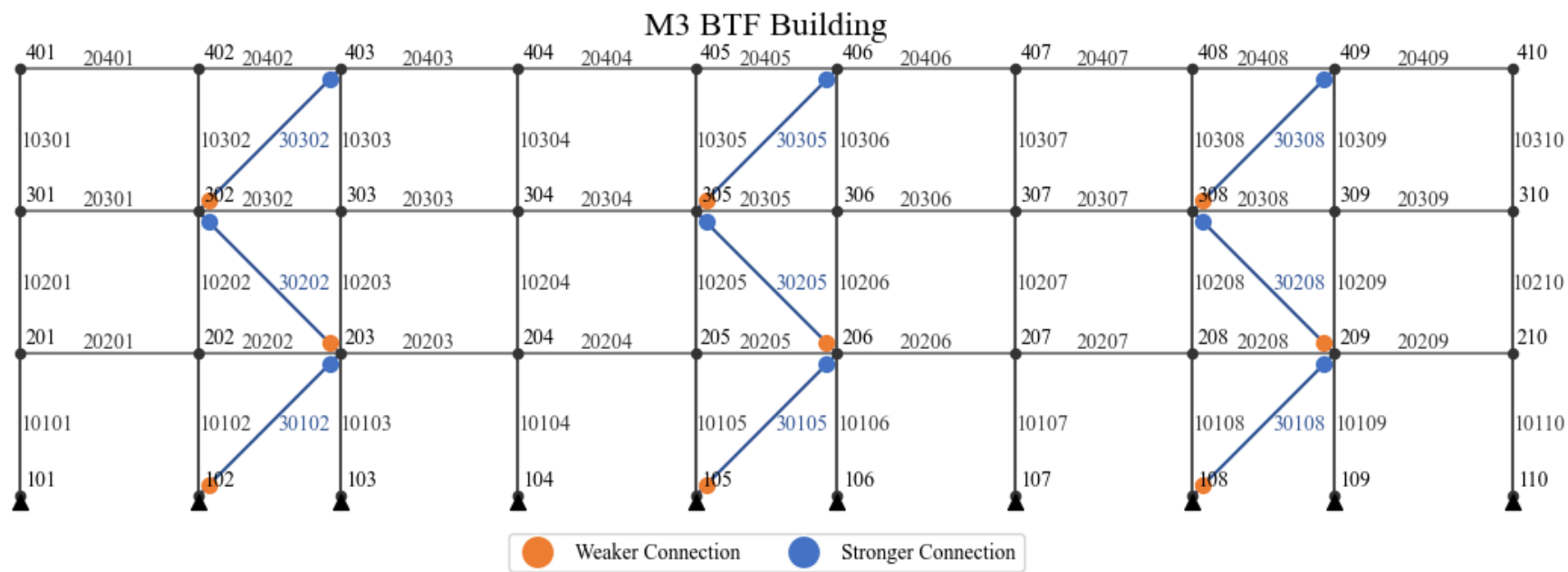


Figure 5.5: M3 Building Prototype - OpenSees Numerical Model

M6 BTF Building

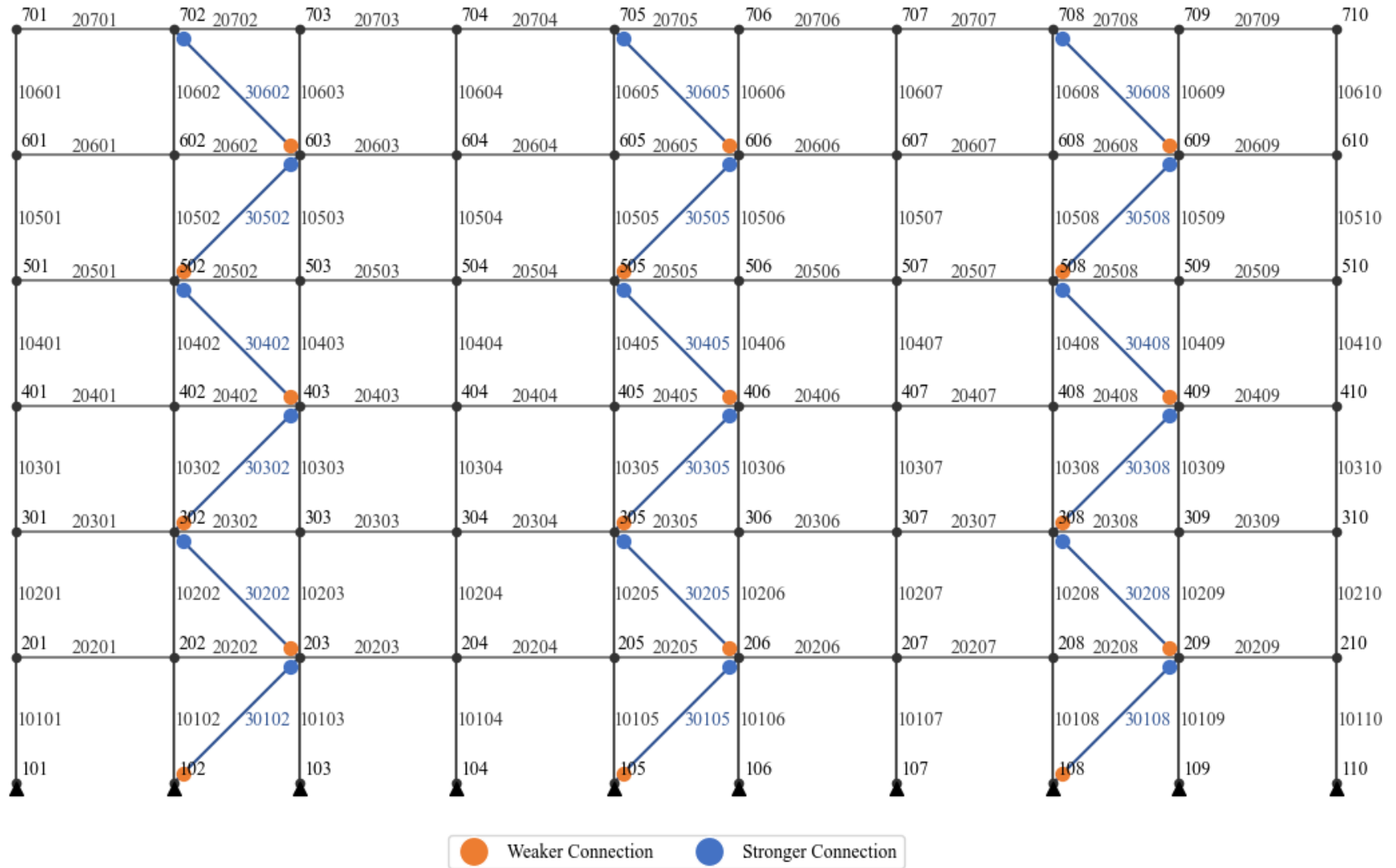


Figure 5.6. M6 Building Prototype - OpenSees Numerical Model

The models were defined by specifying the number of storeys, bays, and their dimensions. Based on this data, node coordinates were generated using a four-digit format (LLGG), where LL is the level and GG is the grid number (e.g., node 204 is located at level 2 and grid 4). Columns were tagged as 1LLGG, beams followed the pattern 2LLBB, and braces (3LLBB) connected duplicate nodes, ensuring the diagonal member remained elastic while zeroLength springs provided nonlinear characteristics at the diagonal member ends. These spring elements were labelled with consecutive tag blocks, enabling a single script line to assign correct spring properties to each storey. By separating linear member behaviour from nonlinear connection mechanics, the model accurately reflects the intended capacity-based design: glulam members remain essentially elastic while brace connections yield and demonstrate self-centering potential depending on their material definitions. These numerical models serve as the basis for the NTHA detailed in Section 5.4 to evaluate interstorey drifts, residual deformations, and peak accelerations.

5.4 Time History Analysis

5.4.1 Analysis Procedure

The analysis procedure for the BTF models consists of a four-step framework developed in OpenSeesPy, which can be found in Appendix C. The first step involved performing a gravity analysis and defining the storey masses. The gravity loads correspond to the total dead loads plus 25% of the specified snow loads, which correspond to the seismic weight, W , determined in Section 4.3. The loads were applied using a linear time series to avoid potential convergence issues. Once the gravity analysis was completed, the loads recorded due to the gravity loads were maintained constant for the subsequent analyses so that P-delta effects are adequately captured.

The second step involved conducting a modal analysis to determine the dynamic characteristics of the prototype buildings. An eigenvalue analysis was performed using an OpenSees built-in function, assembling stiffness and mass matrices based on the previously defined structural models. The first and second periods of the prototype buildings are presented in Table 5.11. The damping coefficient was set to 3% as reported in the study by Popovski et al. (1999). This value is typically adopted in the literature for BTFs with dowel-type connections. The first and second vibration modes are specifically used to determine the damping coefficients, which are then incorporated into the numerical model through the Rayleigh damping formulation. Using the first two modes was adequate because higher mode effects had minimal impact on the structural response (Chen & Popovski, 2021).

Table 5.11: Building Prototype Period Information

Steel Connections			SMA Connections		
Prototype Model	T [s]		Prototype Model	T [s]	
	1st ^a	2nd ^b		1st ^a	2nd ^b
THA-M3-SC	0.58	0.21	THA-M3-NC	0.64	0.24
THA-M6-SC	1.13	0.35	THA-M6-NC	1.28	0.41

^a first mode period

^b second mode period

The third step consisted of defining the dynamic excitation. Ground motion records were imported as processed text files, and simulation durations were set to 2.0 times the record length to capture final phenomena and ensure equilibrium was reached after a period of free vibration. This guarantees an accurate capture of residual deformations post-seismic event. The ground motion records were scaled to align with the target design spectrum at each prototype fundamental period. The details of ground motion selection and scaling procedures will be discussed in Section 5.4.2.

The final step consisted of executing the time history analysis. Representative node interstorey drifts were computed at each time step. The dynamic analysis was carried out using the Newmark- β integrator ($\gamma = 0.5$, $\beta = 0.25$). Convergence was assessed through a displacement-control check, and adaptive convergence strategies, including reduced time increments and alternative algorithms, were employed when necessary. The structural collapse was identified when interstorey drift exceeded 2.5%, aligning with the NBCC (2020) life-safety performance tolerance. If the building were to exceed this limit, the analysis was terminated, and the building was marked as failed. The analysis recorded the displacement and acceleration at the nodes of interest for later post-processing.

5.4.2 Earthquake Information

The ground motion selection and scaling process were determined following the NBCC 2020 Structural Commentary. The period range, T_R , is defined as a period that covers all vibration modes, and it is defined based on the information provided in Equation (5.5) which is determined based on the period at the first mode, T_1 , and the period at 90% mass partition, $T_{90\%}$. The minimum and maximum period limits between the SC and NC buildings in each group (M3 and M6) are close enough to use the same T_R for both buildings. Table 5.12 presents information regarding the T_R for each prototype building.

$$T_{min} = \min\{0.15T_1, T_{90\%}\}, T_{max} = \max\{2T_1, 1.5 s\} \quad (5.5)$$

Table 5.12: Period Range - Target Response Spectrum

Prototype Model	T_1 [s]	$T_{90\%}$ [s]	$T_{\min}$ [s]	$T_{\max}$ [s]	T_R [s]
THA-M3-SC	0.58	0.55	0.09	1.5	0.1 - 1.5
THA-M3-NC	0.64	0.61	0.1	1.5	0.1 - 1.5
THA-M6-SC	1.20	1.13	0.18	2.4	0.2 - 2.5
THA-M6-NC	1.28	1.21	0.19	2.5	0.2 - 2.5

The information regarding the seismic ground motion and site characteristics for a Site Class C in Ottawa was discussed in Section 3.5.2. To summarize, the seismic records for a Site Class C were determined as strike-slip events of magnitude (M) 4.5-6.5. The design spectrum corresponding to a 2% probability of exceedance in 50 years features a PGA of 0.207 g and PGV of 0.147 m/s². Figure 5.7 reproduces this target response spectrum. Two magnitude distance scenarios were considered for record searching: a) $4.5 \leq M < 6$ events b) $M_6 \leq M < M_7$ events, both within 10 km to 30 km from the rupture. Fifteen horizontal components were retrieved from the PEER NGA-West2 database (Ancheta et al., 2014). The selected ground motions are the same records discussed in Section 3.5.2 (refer to Table 3.11).

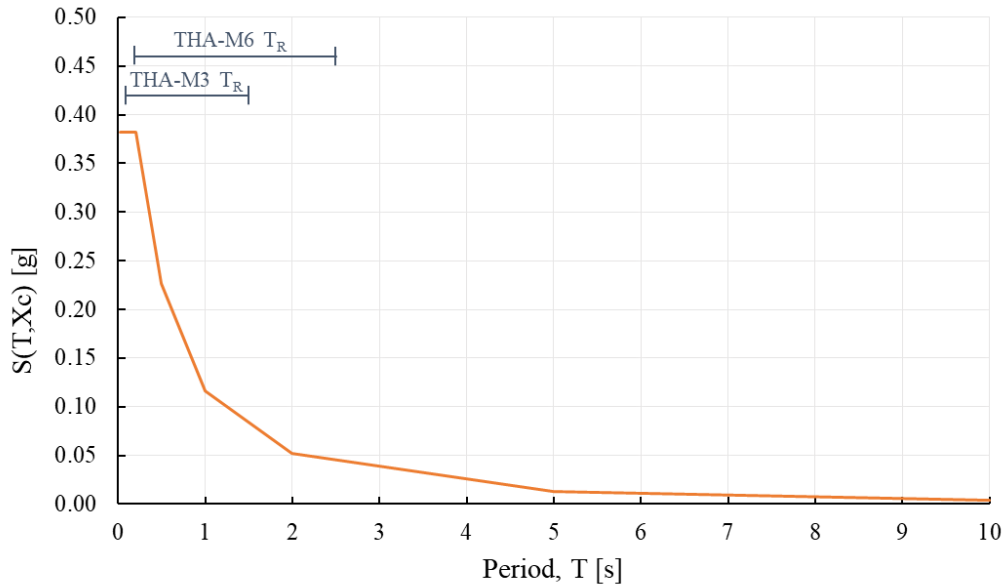


Figure 5.7: Design Spectrum for Ottawa Site Class C and Period Ranges, T_R
Retrieved from NBCC (2020)

The scaling of the ground motion was completed in two stages following Method A defined in the NBCC 2020 Structural Commentary. The first stage involves scaling each individual record so that the average response spectrum equals or exceeds the target spectrum, $S_T(T)$ over T_{RS} . This scaling stage was completed

using the target spectrum in the PEER NGA-West2 database with a geometric mean spectral ordinate, 5% damping ratio, and the arithmetic mean of the suit over T_R . This method finds the optimal amplitude scale factor that minimizes mismatch over the range specified. After this step was completed, a second scaling factor, referred as suit scaling factor, S_F , was applied so that the mean suit spectrum is greater than or equal to 90 percent of $S_T(T)$ throughout T_R . The stage 1 factors for each earthquake record, the suite factor, S_F , (stage 2), and the final scale factors (stage 1 + stage 2) are provided in Table 5.13.

Table 5.13: Ground Motion Records Scaling Factors

		THA-M3 Models	THA-M6 Models
T_R [s]:		0.1 - 1.5	0.2 - 2.5
S_F :		1.28	1.90

No.	ID	Stage 1 Factors	Final Factors	Stage 1 Factors	Final Factors
Eq1	152	1.47	1.88	1.40	2.66
Eq2	222	0.83	1.06	1.00	1.89
Eq3	234	1.57	2.01	1.63	3.10
Eq4	242	1.78	2.28	1.72	3.26
Eq5	1099	1.38	1.76	1.15	2.19
Eq6	4260	1.21	1.55	1.81	3.44
Eq7	4331	1.95	2.49	1.85	3.51
Eq8	9009	0.95	1.22	1.12	2.12
Eq9	9449	1.47	1.88	1.81	3.44
Eq10	9838	1.51	1.93	1.82	3.46
Eq11	450	1.19	1.52	1.06	2.00
Eq12	2709	0.84	1.08	0.66	1.25
Eq13	4284	1.15	1.47	1.29	2.45
Eq14	6057	1.12	1.43	1.05	1.99
Eq15	6878	0.84	1.08	0.84	1.59

It is important to consider that the NBCC (2020) in its commentary suggests that scaling factors are within a range from 0.5 to 4.0 since excessive scaling could mean that the ground motion is not compatible with the source mechanism or seismic hazard level considered. For this scaling process, all ground motions are scaled within this suggested range. Figure 5.8 and Figure 5.9 present the target spectrum, the initial scaled mean (stage 1), and the final suit mean (stage 1 + stage 2). The scaled ground motion records based on the values shown in Table 5.13 are also provided in the figures below.

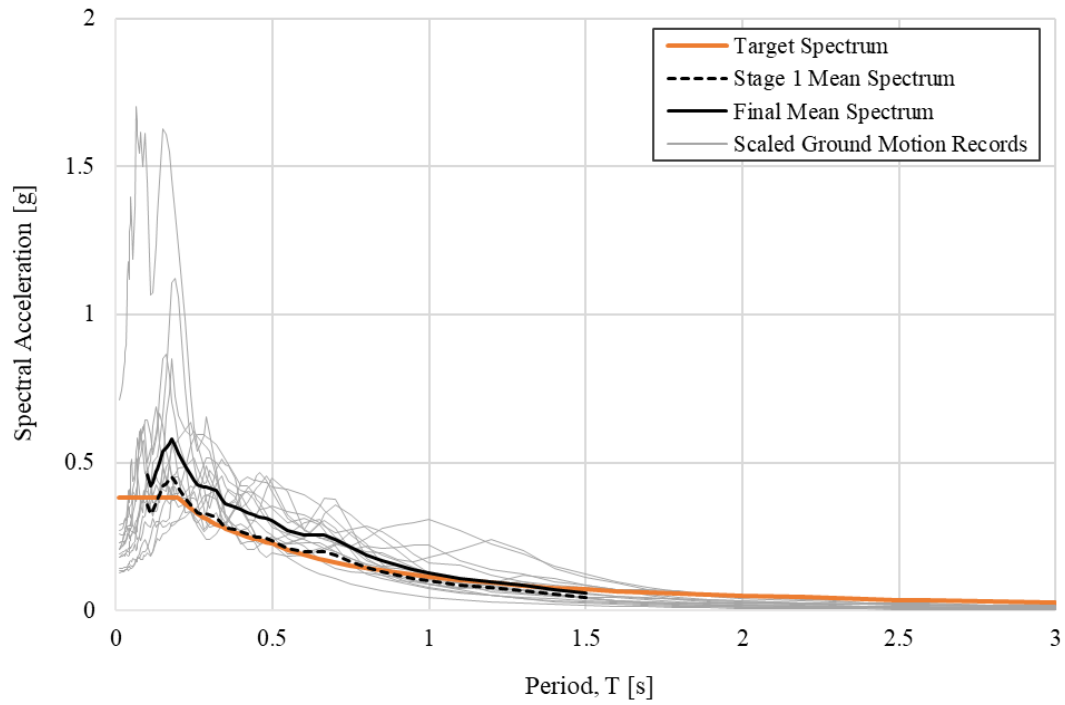


Figure 5.8: M3 Building Prototype –Mean Spectrum of Scaled Ground Motions

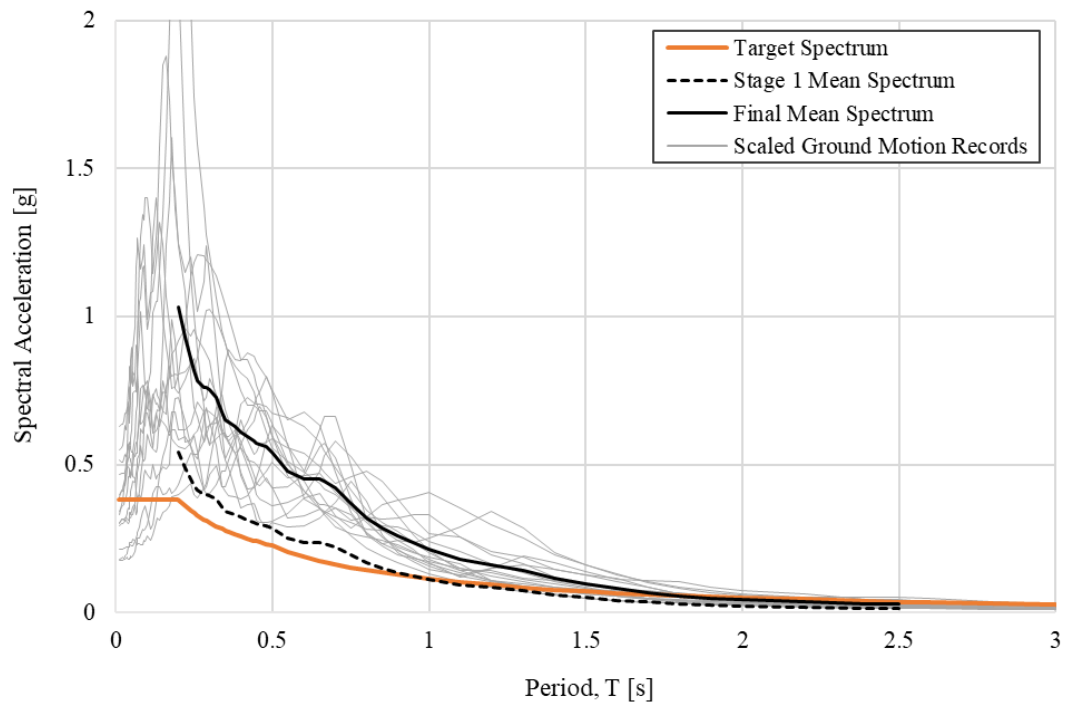


Figure 5.9: M6 Building Prototype –Mean Spectrum of Scaled Ground Motions

CHAPTER 6. RESULTS AND DISCUSSION

6.1 Overview

In chapter 6, the numerical results assess the impact of SMA dowels at a system-level for BTFS in comparison to traditional steel-dowel connection frames. This chapter presents the results for the fundamental periods determined using empirical equations and numerical models. Afterwards, the peak interstorey drifts (or peak drifts) are given for the BTF numerical models, including a discussion regarding the peak interstorey drift limits to assess the results obtained. Consequently, the residual interstorey drifts (or residual drifts) are described, and their results indicate that minimum residual drift in SMA-connected frames, confirming their self-centering ability. The influence of SMA dowel on the peak storey acceleration is also assessed by comparing the findings with steel-connected frames results. Finally, a quantitative comparison is performed by implementing two comparative ratios that measure the effect of SMA-connected frames on the seismic performance of BTFs. Overall, the findings suggest that SMA dowels meaningfully enhance post-earthquake performance without compromising strength demands. This could contribute to future code development or revisions and to further guide the research direction of BTFs with SMA dowel-type connections.

6.2 Fundamental Period

The fundamental natural period, T_a , values from the building prototype models are presented in Figure 6.1, alongside the empirical estimates, T_E , determined following the NBCC 2020 guidelines. The empirical formulation uses a linear function of building height, $0.025 h_n$, where h_n is the building height from base to the mean storey height. The empirical period resulted in 0.30 s for the 12-meter M3 structure and 0.60 s for the 24-meter M6 structure. The fundamental period from the numerical models corresponds to the first-mode period determined using the OpenSees eigen analysis (see Section 5.4). Overall, the values found agree with the general understanding that taller buildings have a longer fundamental period.

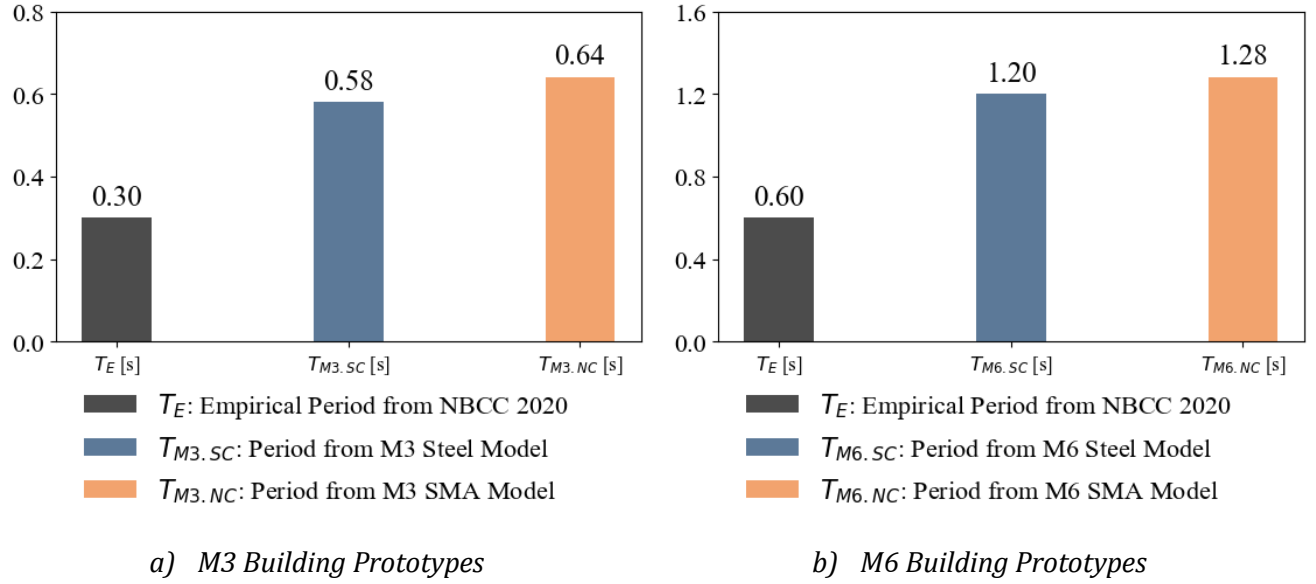


Figure 6.1: Fundamental Periods of Building Prototypes

As shown in the figure above, the numerical periods are about double the empirical values determined for each building prototype. These results are expected since the NBCC 2020 intends to provide a conservative period estimate. Test results by Popovski (2000) and by Chen and Popovski (2021) also found that the fundamental periods of the BTF buildings determined in their studies were longer than their corresponding periods estimated using the NBCC equation.

For both building heights, the fundamental period for the BTF that uses SMA connections was also higher than that of their counterpart with steel connections. The fundamental period increase between the SMA- and steel-connection frames are 10.3% and 6.7% for the M3 and M6 buildings, respectively. The larger difference for M3 reflects the greater influence of brace flexibility at low heights. For both buildings, since the structural members and geometry were modelled identically, the increase in each building height group is attributed to the lower initial stiffness of SMA connections in comparison to the steel connections. At a system-level, the higher period for SMA-connected frames provides insight into the higher flexibility that can be achieved while retaining similar strength capacities.

6.3 Peak Interstorey Drift

6.3.1 M3 Prototype Peak Interstorey Drift

Figure 6.2 and Figure 6.3 present the maximum interstorey drift response of the M3 prototypes. Figure 6.2 shows, for each of the fifteen ground-motion records, the peak drift ratio obtained for Storeys 1 through 3, together with dashed and solid lines that mark the mean values for the steel- and SMA-connected frames,

respectively. Figure 6.3 then shows these average peak interstorey drift as absolute displacement (mm) and drift ratio (%), allowing a direct comparison between connection types.

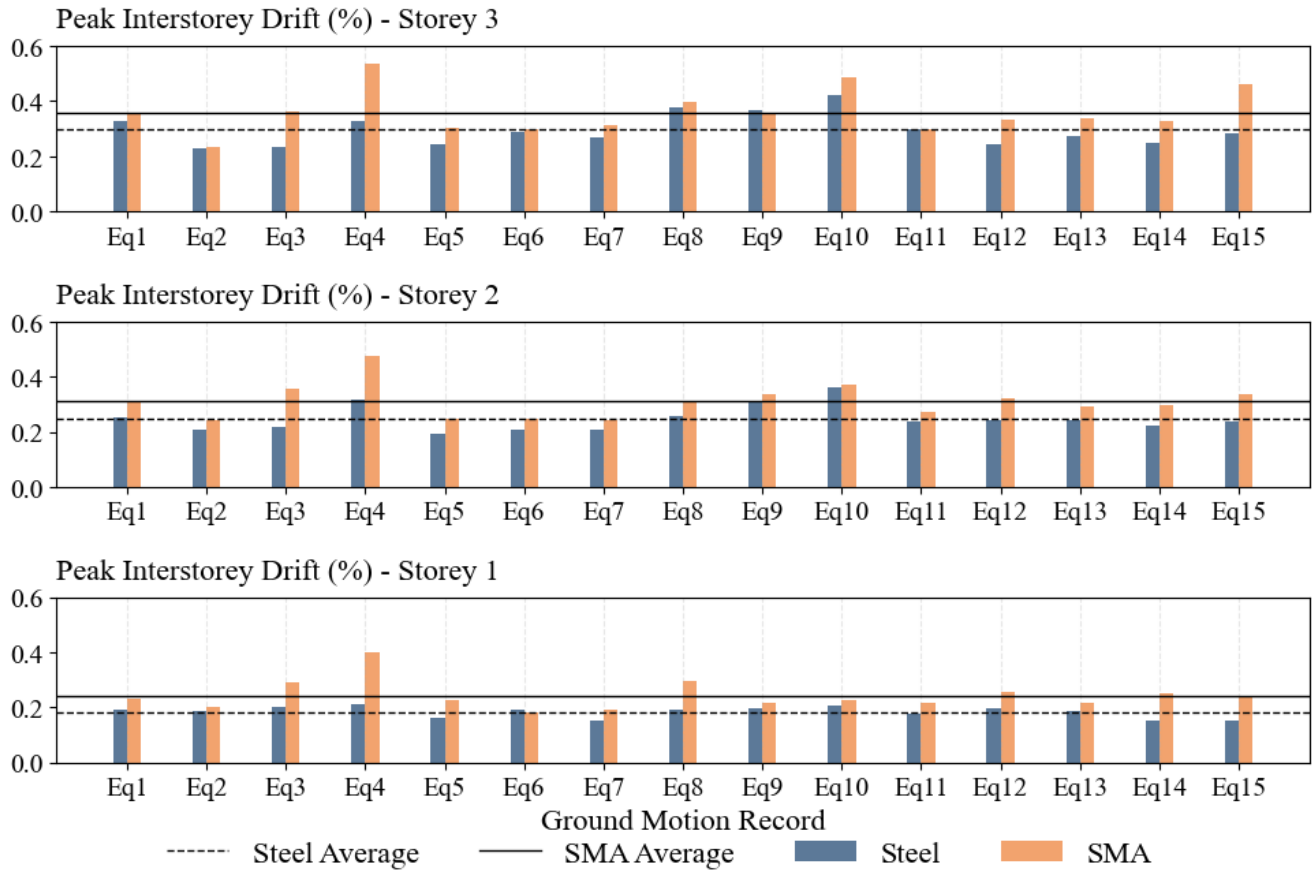
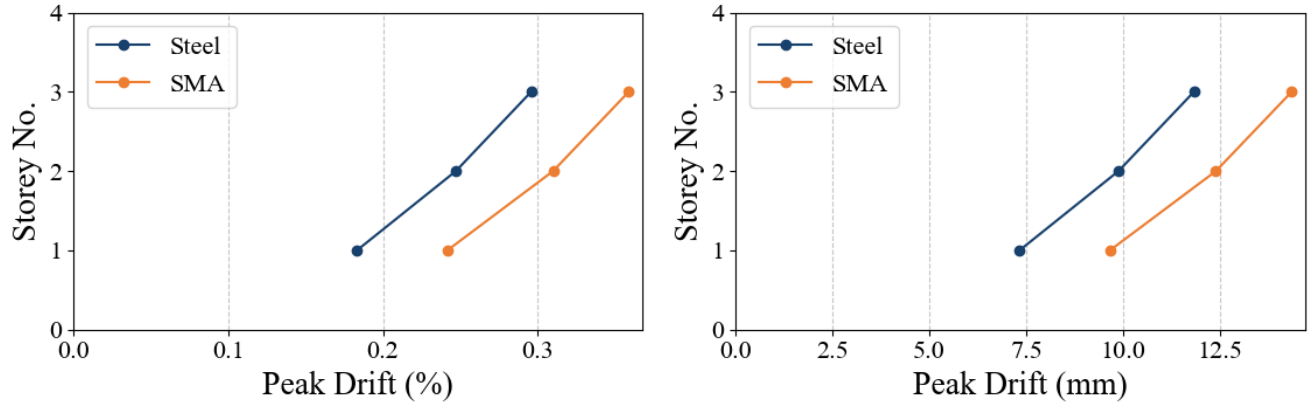


Figure 6.2: M3 Buildings - Peak Interstorey Drift Ratios by Ground Motion Record (Eq)

Figure 6.2 shows various trends to be discussed. First, drift demand climbs with height in both frames; Storey 3 consistently governs, while Storey 1 experiences the least deformation. Second, the SMA prototype exhibits larger drifts for nearly every record, with the difference most pronounced in Eq 4 and Eq 8, where the SMA columns rise well above the steel ones. Third, the spread among records is modest for the steel frame but widens for the SMA case, suggesting greater sensitivity to ground-motion characteristics when lower initial stiffness is present.



a) Peak Interstorey Drift Ratio (%)

b) Peak Interstorey Drift (mm)

Figure 6.3: M3 Buildings – Average Peak Interstorey Drift Results

In Figure 6.3, the peak displacement for both building types is similar since it increases its peak drift as the height increases, meaning that the interstorey drift is higher at the roof level (storey 3) in both M3 building types. The average peak drift at Storey 1 rises from 0.183% (7.33 mm) in M3-Steel to 0.242% (9.67 mm) in M3-SMA, a 32% increase. At mid-height (storey 2), the values move from 0.247% (9.89 mm) to 0.310% (12.40 mm), representing about a 26% increment. The most significant increase is shown at storey 3, the drift ratio increases from 0.296% (11.84 mm) to 0.359% (14.36 mm), a 21% increase. Averaged across the three storeys, the M3 building frame with SMA connections undergoes roughly 26% greater peak displacement than its steel counterpart, a trend consistent with the lower initial stiffness of SMA connections.

6.3.2 M6 Prototype Peak Interstorey Drift

Similar to the figures for the M3 building prototypes, Figure 6.4 and Figure 6.5 present the peak interstorey drift results for the M6 prototypes. Figure 6.4 plots, for each of the fifteen ground-motion records, the maximum drift ratio recorded at Storeys 1 through Storey 6. Dashed lines denote the steel average, while solid lines show the SMA average. Figure 6.5 reiterates those storey-mean values as absolute displacements (mm) alongside the corresponding drift ratios (%), accentuating the difference between connection types.

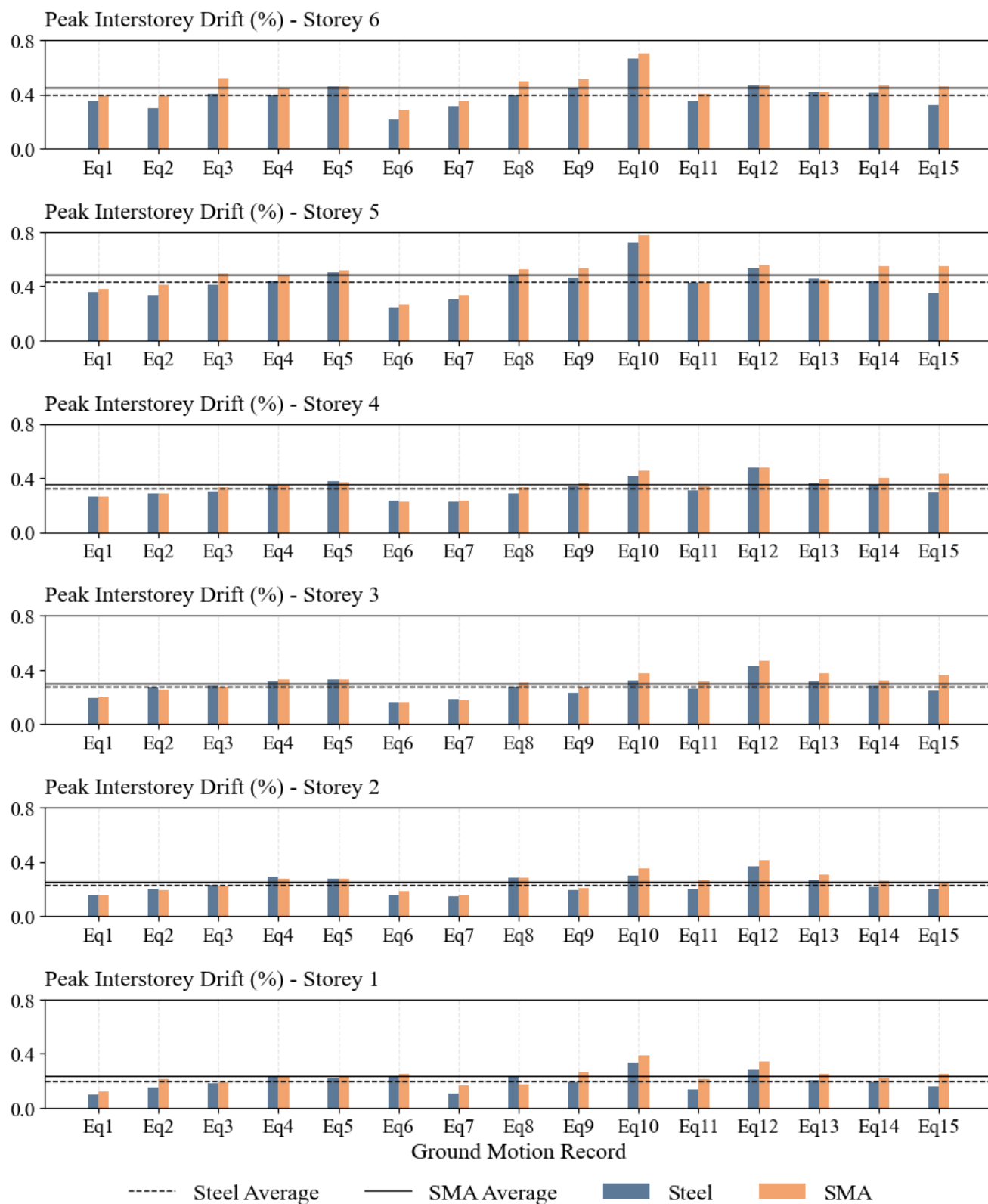
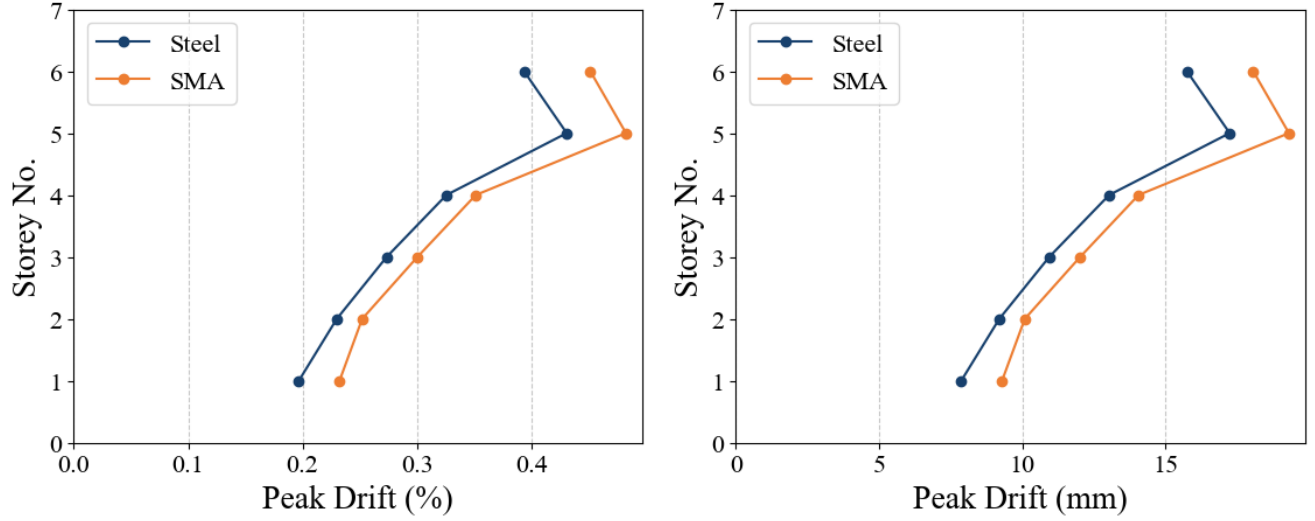


Figure 6.4: M6 Buildings – Peak Interstorey Drift Ratios by Ground Motion Record (Eq)

The interstorey drift grows steadily from the base to Storey 5 before tapering slightly at the roof. This indicates that the penultimate level is the most flexible segment under the adopted brace and mass configuration. The SMA-connected prototype records larger peak drifts than the steel frame in almost every ground-motion case. Record-to-record scatter remains tight for the steel system but broadens for SMA, implying greater sensitivity to ground-motion characteristics when initial stiffness is reduced.



a) Peak Interstorey Drift Ratio (%)

b) Peak Interstorey Drift (mm)

Figure 6.5: M6 Buildings – Average Maximum Interstorey Drift Results

As shown in Figure 6.5, every level of the SMA frame registers greater peak drifts than the steel frame, an outcome that aligns with the lower initial stiffness of the SMA connections. At Storey 1, the mean demand rises from 0.197% (7.87 mm) with steel to 0.232% (9.29 mm) with SMA, roughly an 18% increase. The gap narrows at mid-height to increments of about 10% at Storeys 2 and 3 and 8% at Storey 4. The difference then widens again near the top levels with Storey 5 reaching 0.483% (19.31 mm) for SMA against 0.431% (17.33 mm) for steel (around 12% higher), and Storey 6 shows 0.452% (18.06 mm) for SMA compared to 0.394% (15.74 mm) for steel, which is about a 15% increment. Averaging across all six levels, SMA connections reach a peak drift of 0.345%, while steel models average 0.308%, yielding an overall increase of roughly 12%.

6.3.3 Peak Interstorey Drift Limits

This subsection evaluates the maximum interstorey drift results presented against calculated drift limits. These limits were determined using the spring theory equations provided in Section 2.3.2, accounting for the stiffness of the connections, timber members, connection yield strengths, and brace geometry detailed

in Section 4.4.2. The resulting drift limits, summarized in Table 6.1 and Table 6.2, vary by storey due to differences in connection capacities and timber cross-sectional dimensions. The peak-to-limit drift ratios have been plotted in Figure 6.6 to show the differences between the SMA- and steel-connected frames for both building prototypes.

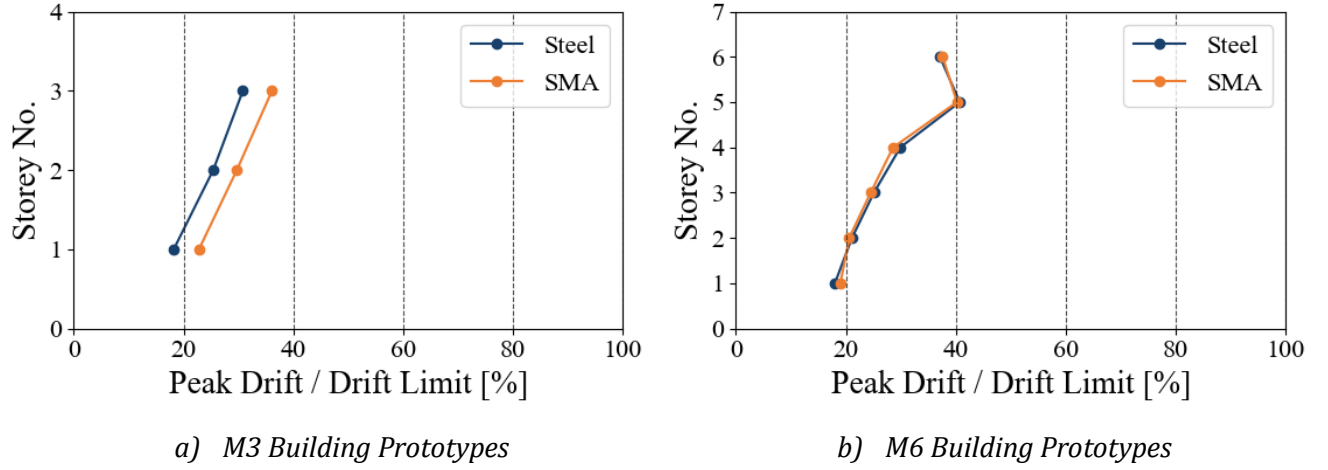


Figure 6.6: Interstorey Peak Drift to Drift Limit Ratio

For the M3 building prototype, Figure 6.6 indicates that the calculated drift limits for each storey range from approximately 0.96% to 1.06% across the three storeys. The average peak interstorey drift ratios obtained from the analyses (Section 5.4.2) range from 0.183% to 0.359%, representing between 18% and 36% of the calculated limits. Notably, the drift limits for SMA-connected frames are higher due to their lower initial stiffness; however, these frames also exhibit higher peak drift ratios. Consequently, the peak-to-limit ratios for SMA frames are slightly higher than steel-connected frames. These ratios indicate sufficient reserve capacity against the theoretical drift limits for both connection types, suggesting that the M3 prototype buildings can comfortably accommodate the seismic demands for the selected site conditions.

Table 6.1: M3 Buildings – Interstorey Peak Drift to Drift Limit Ratio Summary

Storey	Material	Connection Name	Peak Drift (%)	Drift Limit (%)	Peak/Limit (%)
[1]	Steel	SC-3/8B-20-4/5	0.1832	1.00	18.2
	SMA	NC-3/8B-20-4/5	0.2417	1.06	22.8
[2]	Steel	SC-3/8B-15-3/5	0.2472	0.97	25.4
	SMA	NC-3/8B-15-3/5	0.3101	1.04	29.7
[3]	Steel	SC-3/8B-9-3/3	0.2959	0.96	30.8
	SMA	NC-3/8B-9-3/3	0.3590	0.99	36.1

Similarly, the M6 prototype (Table 6.2) presents calculated drift limits varying from 1.06% at the lower storeys to 1.23% at the upper storeys. Average peak interstorey drift ratios observed in the analyses range

from 0.197% to 0.483%, equating to approximately 18% to 41% of the corresponding drift limits. As with the M3 prototype, drift limits for the SMA-connected frames are higher due to their lower initial stiffness, yet these frames also demonstrate higher peak drifts, resulting in elevated peak-to-limit ratios. These results place the M6 building within a moderate range of reserve capacity, closer to the limits compared to the M3 building, but still demonstrating acceptable performance.

Table 6.2: M6 Buildings – Interstorey Peak Drift to Drift Limit Ratio Summary

Storey	Material	Connection Name	Peak Drift (%)	Drift Limit (%)	Peak/Limit (%)
[1]	Steel	SC-1/2B-16-4/4	0.1967	1.09	18.0
	SMA	NC-1/2B-16-4/4	0.2323	1.23	19.0
[2]	Steel	SC-1/2B-16-4/4	0.2299	1.09	21.1
	SMA	NC-1/2B-16-4/4	0.2521	1.23	20.6
[3]	Steel	SC-1/2B-15-3/5	0.2735	1.09	25.1
	SMA	NC-1/2B-15-3/5	0.3004	1.23	24.5
[4]	Steel	SC-1/2B-15-3/5	0.3257	1.09	29.9
	SMA	NC-1/2B-15-3/5	0.3514	1.23	28.6
[5]	Steel	SC-1/2B-9-3/3	0.4309	1.06	40.7
	SMA	NC-1/2B-9-3/3	0.4827	1.20	40.2
[6]	Steel	SC-1/2B-9-3/3	0.3936	1.06	37.1
	SMA	NC-1/2B-9-3/3	0.4515	1.20	37.6

These findings indicate that the current NBCC 2020 drift limit of 2.5% is overly conservative for BTFs such as those presented here, as the maximum estimated peak drift in this study remains around 1% or less across all storeys, significantly below the NBCC threshold. Moreover, considering that the M3 prototype (12 metres height) meets the NBCC 2020 Seismic Category 3 height restriction of 15 m for limited ductility BTFs, and that the M6 prototype (24 metres height) exceeds this limit, the observed moderate drift-to-limit ratios in the taller M6 building suggest potential feasibility in raising the NBCC height restrictions.

6.4 Residual Interstorey Drift

Section 6.3 demonstrated that BTFs with SMA connections experienced higher peak interstorey drifts compared to steel-connected frames. Figure 6.7 through Figure 6.10 illustrate the residual interstorey drift outcomes for both building prototypes, providing insight into the effectiveness of SMA connections in restoring structural alignment after seismic events.

6.4.1 M3 Prototype Residual Interstorey Drift

Figure 6.7 and Figure 6.8 display the residual interstorey drift performance of the M3 building prototypes. Figure 6.7 shows the residual drift ratios recorded at each of the fifteen ground-motion records for Storeys 1 through 3, with dashed and solid horizontal lines representing the mean values for steel- and SMA-connected frames, respectively. Figure 6.8 presents the storey-averaged residual drift ratios (%) and absolute displacements (mm), following the format used previously for peak drift.

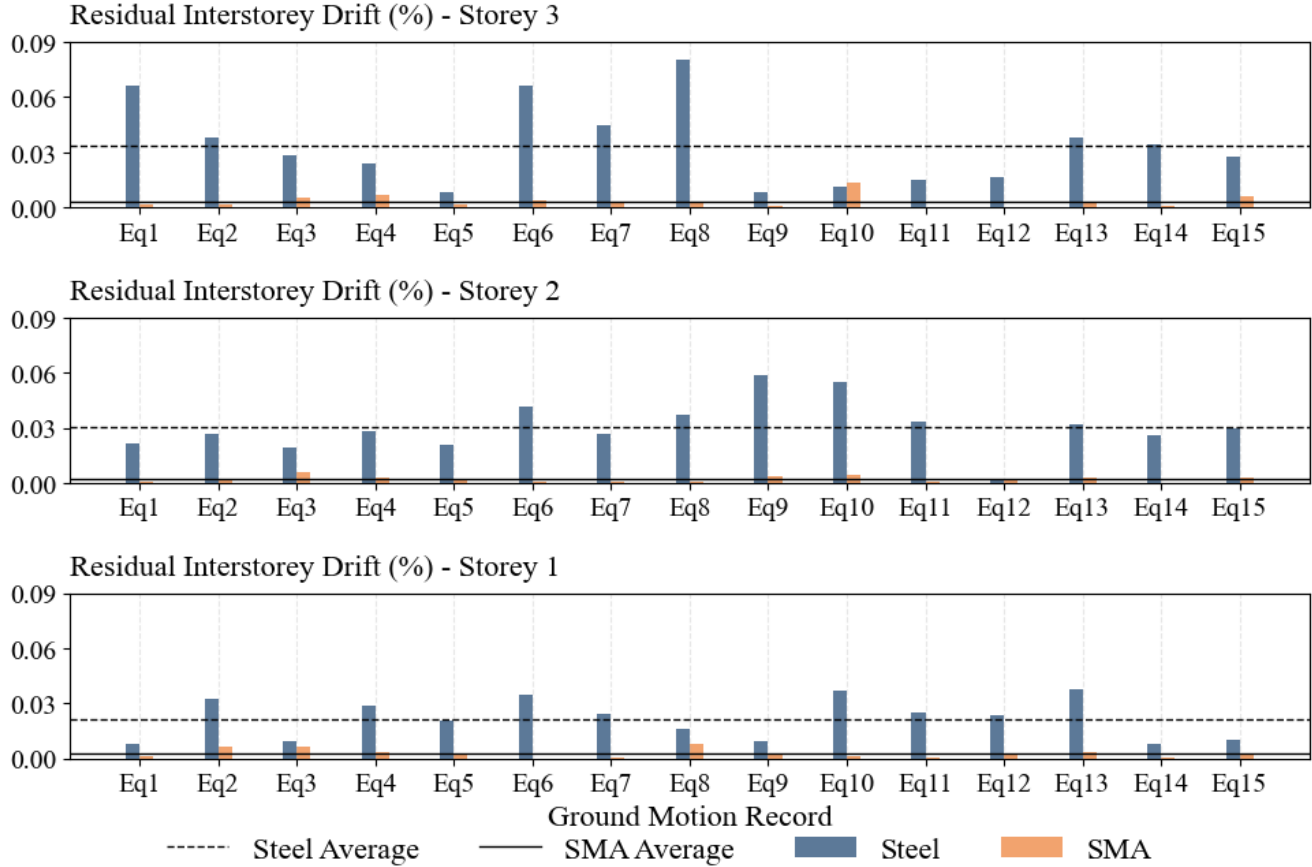


Figure 6.7: M3 Buildings – Residual Drift Ratios by Ground Motion Record (Eq)

Throughout all ground motions, residual interstorey drifts for steel-connected BTFs ranged from 0.01% to 0.09%, whereas the SMA-connected frames exhibited residual drifts near zero. Even under the most demanding records (Eq8 and Eq10), the building with SMA connections showed negligible permanent deformation, whereas the steel-connected frames consistently retained noticeable permanent offsets at every storey. Consequently, record-to-record variability was primarily influenced by the response of steel

connections, highlighting how the plastic deformation of conventional steel dowels translates into permanent interstorey deformations.

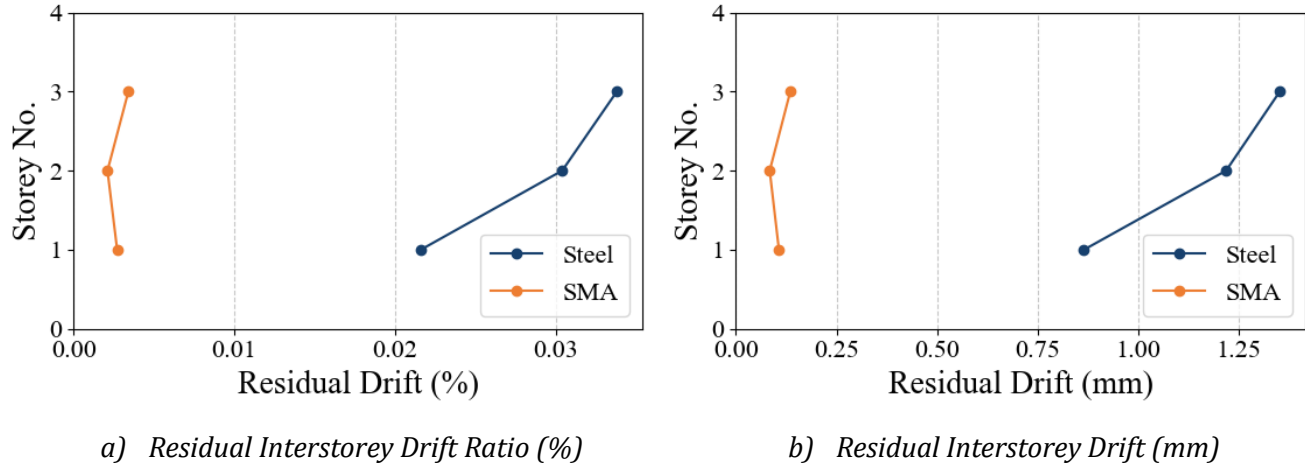


Figure 6.8: M3 Buildings – Average Residual Interstorey Drift Results

The average residual drifts for steel-connected frames followed a trend similar to their peak drift ratios, as permanent deformation increased with height. In contrast, SMA-connected frames maintained a constant and minimal residual drift profile across all storeys. At Storey 1, residual drift for steel-connected frames was 0.0216%, roughly eight times larger than the frames with SMA connections (0.0027%). The difference increased notably at Storey 2, where frames with steel connections showed residual drift of 0.0304%, more than 14 times that of the SMA-connected frames (0.0021%). At Storey 3, the residual drift for steel-connected frames reached 0.0338%, about ten times greater than the SMA-connected frames (0.0034%). The minimal residual drift ratios of SMA-connected frames demonstrated the strong self-centering capability of SMA connections.

6.4.2 M6 Prototype Residual Interstorey Drift

Figure 6.9 expands the residual drift ratio analysis to the M6 prototype, showing individual record responses at each storey, with dashed and solid lines indicating average values for steel- and SMA-connected frames, respectively. Figure 6.10 summarizes these storey-average drift ratios (%) and also presents them in absolute displacements (mm).

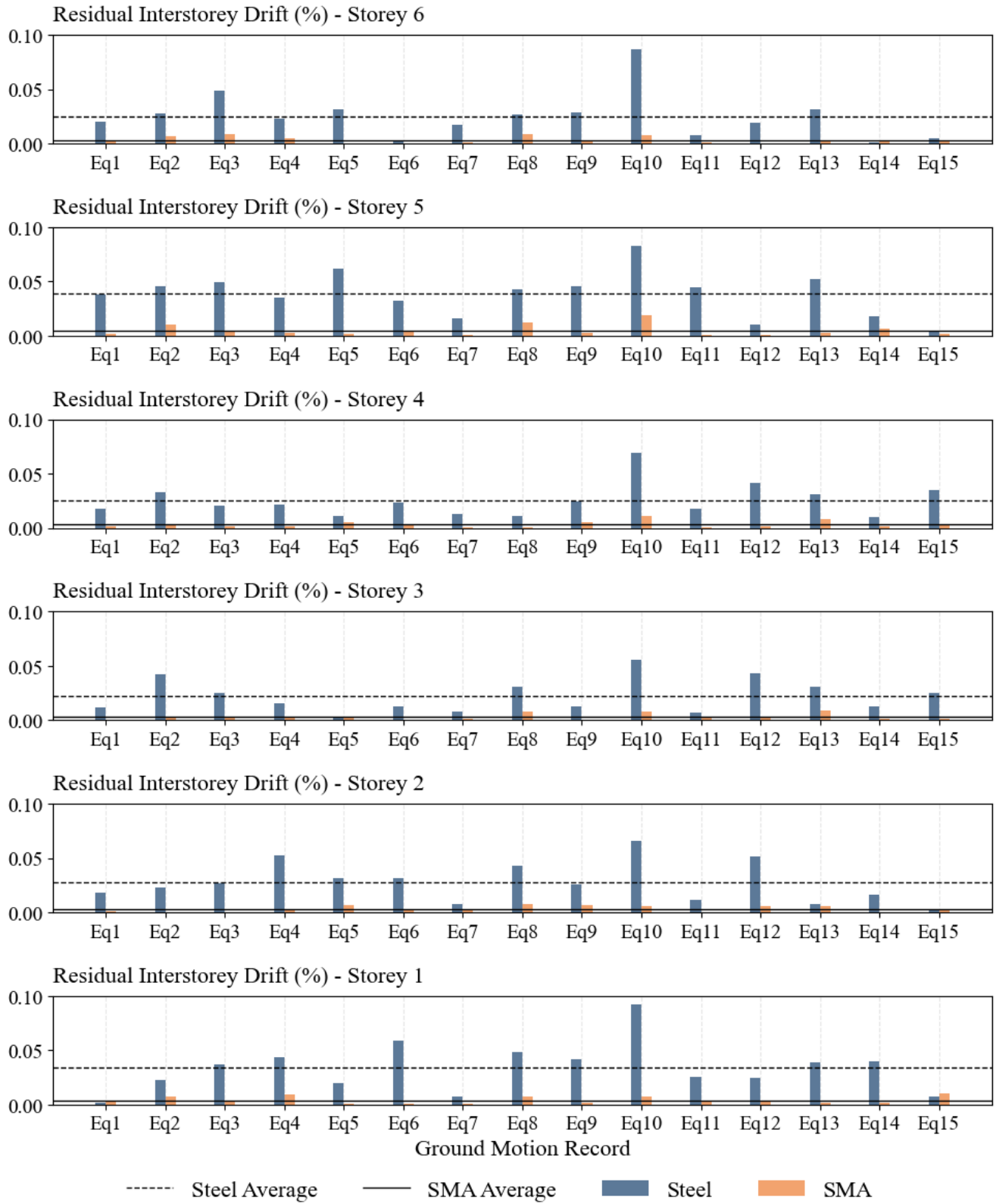


Figure 6.9: M6 Buildings – Residual Interstorey Drift Ratios by Ground Motion Record (Eq)

Residual interstorey drifts for the steel-connected frame consistently exceeded those of the SMA-connected frame for almost every ground motion record. These residual drifts were most pronounced at Storeys 1, 5, and 6. Consistent with the M3 results, Eq10 was again the most demanding record, resulting in residual drifts near 0.09% across the steel-connected frame, whereas SMA-connected frames remained below 0.02%. Overall, these observations reinforced the effectiveness of SMA connections in substantially reducing residual drift and limiting record-to-record variability compared to traditional steel fasteners.

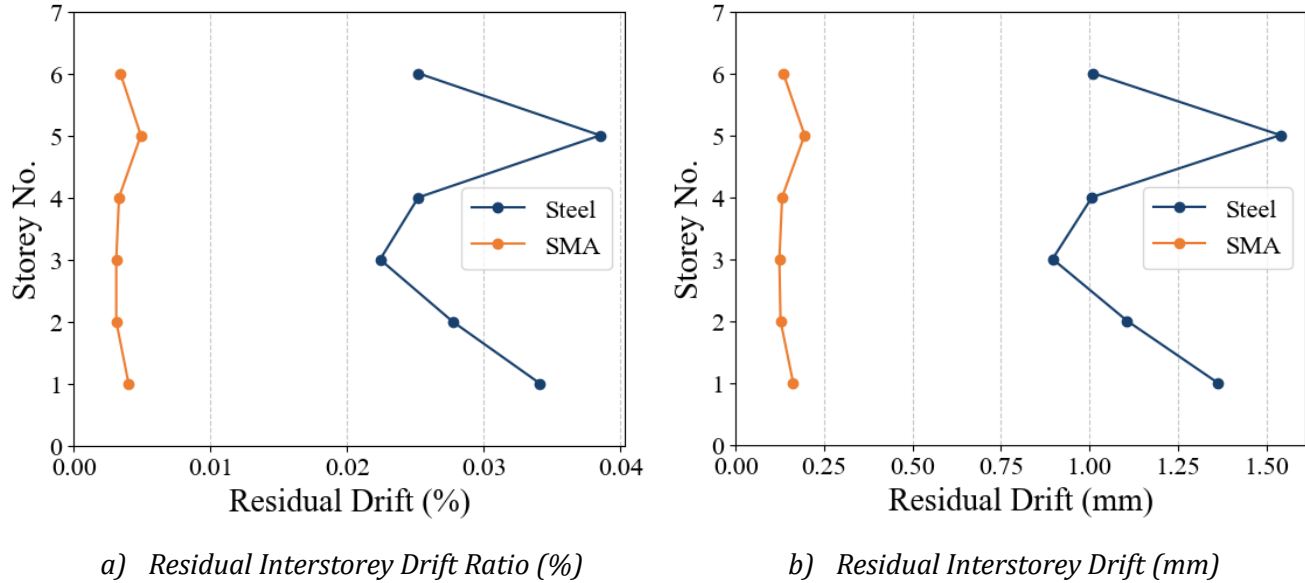


Figure 6.10: M6 Buildings – Average Residual Interstorey Drift Results

As shown in Figure 6.10, average residual drifts for steel-connected frames in the M6 prototype varied notably across storeys, ranging from 0.022% to 0.042% (0.89–1.67 mm). SMA-connected frames, however, maintained a stable and minimal residual drift pattern below 0.005% (0.2 mm) for all storeys. At storey 1, steel-connected frames exhibited residual drift of 0.0341%, approximately 8.5 times greater than the SMA-connected frame (0.004%). Residual drift values in SMA-connected frames remained consistently low at around 0.003% for intermediate storeys 2 to 4, whereas steel-connected frames reached 0.028%, 0.022%, and 0.025%- representing increases of about 8.9, 7.2, and 7.6 times over SMA-connected frames for storeys 2, 3, and 4, respectively. Storey 5 recorded the highest residual drifts for both connection types (0.039% steel vs. 0.005% SMA), maintaining a similar steel-to-SMA ratio of about 8:1. At the roof level (storey 6), residual drift in steel-connected frames reduced slightly to 0.025%, approximately 7.4 times greater than the 0.0034% observed for SMA-connected frames.

6.5 Peak Storey Acceleration

Sections 6.3 and 6.4 showed that SMA-connected BTFs experienced higher peak interstorey drifts but significantly reduced residual drifts compared to steel-connected frames. Alongside drift performance, peak floor accelerations are crucial, particularly for acceleration-sensitive non-structural components. Figure 6.11 and Figure 6.12 summarize the average peak floor acceleration results for the M3 and M6 building prototypes, illustrating how SMA and steel connections affect the dynamic response of BTFs.

6.5.1 M3 Prototype Peak Storey Acceleration

Figure 6.11 displays the storey-average peak floor accelerations, expressed in units of gravity (g), for the M3 prototype buildings. The accelerations were computed as the mean of peak floor accelerations from the full suite of fifteen ground-motion records for each storey level. Results indicate that SMA-connected frames consistently exhibit lower peak accelerations than steel-connected frames at every storey, with reductions of 4% at the roof (storey 3), 5% at storey 2, and 13% at the first storey. Specifically, SMA frames presented mean accelerations of 0.33 g, 0.41 g, and 0.53 g at Storeys 1, 2, and 3, respectively, whereas steel frames recorded higher accelerations of 0.38 g, 0.43 g, and 0.55 g for the corresponding storeys.

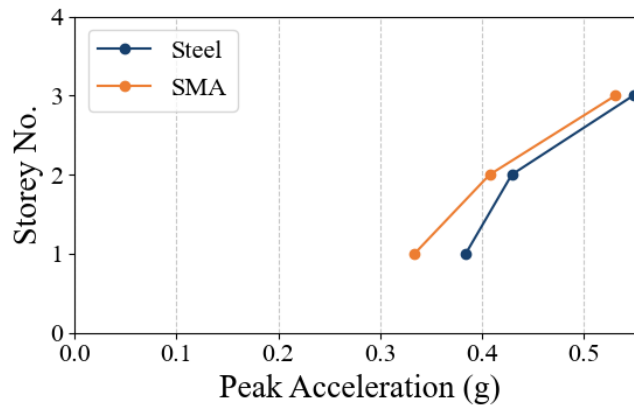


Figure 6.11: M3 Buildings – Average Peak Acceleration

The reduced floor accelerations observed in the SMA-connected frames can be attributed to their lower initial stiffness and consequently longer fundamental periods, shifting their structural response toward lower spectral acceleration ordinates. Despite the higher peak drifts for SMA-connected frames (Section 6.3), their softer dynamic response reduced inertial forces and floor accelerations.

6.5.2 M6 Prototype Peak Storey Acceleration

Figure 6.12 shows the corresponding peak acceleration results for the M6 prototype buildings. As in the M3 cases, peak floor accelerations were determined by averaging all fifteen ground-motion records for each storey level. Similarly, SMA-connected frames consistently exhibited lower accelerations than steel-connected frames across all storeys. At Storey 1, the mean peak floor acceleration decreased from 0.55g in steel-connected frames to 0.51g for SMA-connected frames, representing about an 8% reduction. This difference widened slightly at intermediate storeys, reaching approximately 14% at Storey 4, where accelerations were 0.55g for steel versus 0.50g for SMA. This difference reduces at the roof level (storey 6), where SMA-connected frames recorded a peak acceleration of 0.63g, approximately 2% lower than the 0.66g acceleration measured for steel-connected frames.

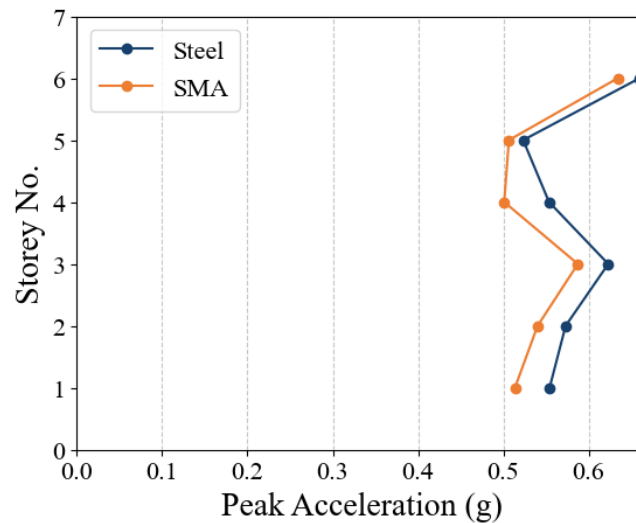


Figure 6.12: M6 Buildings – Average Peak Storey Acceleration

Consistent with the findings for M3 buildings, the longer fundamental periods and lower stiffness of SMA-connected frames contributed to reduced spectral acceleration demands and lower inertial forces transmitted to the floors. Even though SMA-connected frames allowed higher peak interstorey drifts and significantly reduced residual drifts (Sections 5.4.2 and 5.4.3), they also resulted in lower floor accelerations, effectively balancing drift flexibility with decreased acceleration impacts. This indicates that SMA dowel-type connections can enhance seismic resilience not only by minimizing permanent deformation but also by reducing demands on acceleration-sensitive non-structural elements.

6.6 Quantitative Comparison

To consolidate the findings from Sections 6.3 to 6.5, a quantitative evaluation was carried out using two comparative ratios. The first one is the SMA Influence Ratio (SMA-I Ratio), which expresses the percentage change of the SMA frame response relative to the steel frame baseline (equation (6.1)), where positive values indicate increases and negative values represent reductions. The second ratio is the residual-to-peak drift ratio (RP Ratio), which measures the proportion of permanent deformation in relation to the maximum displacement as a percentage value (see equation (6.2)). The RP Ratio results of both buildings have been summarized in Table 6.3 and Table 6.4 for the M3 and M6 prototypes, respectively.

$$SMA\ Influence\ Ratio = SMA.I\ Ratio = -\frac{SR_{steel\ frame} - SR_{SMA\ frame}}{SR_{steel\ frame}} \times 100 \quad (6.1)$$

where, $SR_{steel\ frame}$ = seismic response quantity for steel frame
 $SR_{SMA\ frame}$ = seismic response quantity for SMA frame

$$RP\ Ratio = \frac{Residual\ Drift}{Peak\ Drift} \times 100 \quad (6.2)$$

Table 6.3: M3 Buildings – Quantitative Analysis Comparison

Storey	Material	Peak Drift (%)	SMA Inf (%)	Residual Drift (%)	SMA Inf (%)	Acc. (g)	SMA Inf (%)	RP Ratio (%)
[1]	Steel	0.183	+31.9	0.0216	-87.5	0.38	-13.1	11.8
	SMA	0.242		0.0027		0.33		1.1
[2]	Steel	0.247	+25.4	0.0304	-93.1	0.43	-5.1	12.3
	SMA	0.310		0.0021		0.41		0.7
[3]	Steel	0.296	+21.3	0.0338	-89.9	0.55	-3.2	11.4
	SMA	0.359		0.0034		0.53		0.9

For the M3 building prototype, SMA-connected frames exhibited peak drift ratios approximately 21% to 32% higher than their steel counterparts. This increase aligns with the lower initial stiffness and greater flexibility inherent in SMA connections. However, despite this increased peak drift, SMA connections significantly reduced residual drifts by approximately 88% to 93%. This substantial reduction in residual drift is attributed to the superelastic self-centering behaviour of SMA, contrasting greatly with the steel connections, which typically retain permanent deformations after yielding. Therefore, SMA connected frames recorded RP ratios from 0.7 to 1.1 percent, representing roughly one-tenth of the values measured for steel frames, whose RP ratios fell between 11.4% and 12.3%.

Floor acceleration responses further highlighted the beneficial behaviour of SMA-connected frames, with peak floor accelerations consistently lower than those observed in steel-connected frames. Specifically, SMA connections reduced peak accelerations by approximately 3% to 13%, indicating a clear advantage in limiting seismic demands on non-structural components. The pronounced reduction at lower floors (13% at Storey 1) suggests a significant contribution from the flexibility of SMA in reducing force transmission through the structure, whereas the smaller reduction at upper floors (3% at Storey 3) aligns with the relatively more flexible response of upper-level bracing.

Table 6.4: M6 Buildings – Quantitative Analysis Comparison

Storey	Material	Peak Drift (%)	SMA Inf (%)	Residual Drift (%)	SMA Inf (%)	Acc. (g)	SMA Inf (%)	RP Ratio (%)
[1]	Steel	0.197	+18.1	0.0341	-88.3	0.55	-7.3	17.3
	SMA	0.232		0.0040		0.51		1.7
[2]	Steel	0.230	+9.7	0.0277	-88.8	0.57	-5.7	12.0
	SMA	0.252		0.0031		0.54		1.2
[3]	Steel	0.274	+9.8	0.0224	-86.2	0.62	-5.7	8.2
	SMA	0.300		0.0031		0.59		1.0
[4]	Steel	0.326	+7.9	0.0252	-86.9	0.55	-9.5	7.7
	SMA	0.351		0.0033		0.50		0.9
[5]	Steel	0.431	+12.0	0.0252	-87.3	0.52	-3.2	8.9
	SMA	0.483		0.0033		0.51		1.0
[6]	Steel	0.394	+14.7	0.0252	-86.5	0.66	-3.9	6.4
	SMA	0.452		0.0033		0.63		0.8

In the taller M6 prototype, similar quantitative observations were determined, although differences between SMA and steel frames were less pronounced, given the inherently greater flexibility of taller buildings. Peak drift ratios increased by approximately 8% to 18% in SMA frames relative to steel frames, due to the lower initial stiffness of SMA connections as highlighted in the M3 building discussion. However, SMA connections reduced residual drift by about 87% to 90%, confirming stable superelastic recentering performance despite building heights. As a result, RP Ratios dropped significantly from around 8% to 10% in steel-connected frames to approximately 0.8% to 1.7% in SMA-connected frames. In addition, floor accelerations in the M6 prototype also exhibited a consistent reduction of roughly 3% to 10% with SMA connections. Acceleration reductions became marginally smaller at higher storeys, reflecting the coupled effects of increased building flexibility, damping, and dynamic characteristics at greater heights.

In addition, comparing the M3 and M6 prototypes provides valuable insights into how building height influences seismic performance when using SMA connections. The increase in peak drift attributed to SMA connections was notably higher in the M3 prototype (approximately 26%) compared to the M6 prototype (around 12%). This observation indicates that the incremental flexibility (larger peak interstorey drifts)

exhibited SMA-connected frames tends to reduce as building height increases. Nevertheless, both prototypes consistently demonstrated significant reductions in residual drift (approximately 90%) and peak floor accelerations (approximately 3–13 %). These consistent outcomes confirm that SMA connections effectively maintain their self-centering ability irrespective of building height.

CHAPTER 7. CONCLUSIONS AND FUTURE WORK

7.1 Conclusions

The research program was comprised of three interconnected modelling phases designed to evaluate the seismic performance of BTF buildings equipped with SMA dowel-type connections in comparison to steel fastener connections. In the first phase, uniaxial spring models were calibrated to match the experimentally-observed hysteresis curves of both steel and SMA connections. A modelling framework was also presented to develop brace-level and frame-level numerical models, validated using theoretical methods. Subsequently, pushover and time history analyses of multi-storey models were defined to assess the self-centering potential of SMA connections observed at the connection-level. For the second phase, prototype buildings were designed considering ductility factor guidelines for the connection detailing process and following NBCC and CSA requirements for a Site Class C Ottawa site. In the final phase, four archetype frames were subjected to fifteen spectrum-scaled ground motions through a nonlinear time-history procedure, enabling direct comparison between SMA- and steel-connected systems. The analytical findings showed that SMA dowels maintained peak drift within code limits while reducing residual drift by around 90%, highlighting their self-centering ability for structures in moderate seismic regions in Canada.

The following conclusions can be drawn from this study:

- The OpenSees analytical framework reproduces the nonlinear response and self-centering behaviour seen in connection-level tests, allowing system-level models to evaluate the seismic performance of full building prototypes. This approach therefore closes the gap between laboratory observations at the connection scale and their practical application in real-world structures.
- In contrast to the results for steel-connected frames, the SMA fasteners raised peak interstorey drifts by 21-32% in the three-storey frame and 8-18% in the six-storey frame. Nevertheless, these higher peak interstorey drifts in SMA-connected frames also show significantly reduced residual drifts compared to steel-connected frames. Comparison between residual drifts for SMA- and steel-connected frames shows an average reduction of 90% and 87% for the three-storey and six-storey buildings, respectively.
- The peak floor accelerations dropped by 3% to 13% when SMA connections were used, which could result in lower non-structural damage and improved occupant comfort. The SMA-connected frames

consistently exhibited lower accelerations compared to steel-connected frames across all storeys, leading to reduced spectral acceleration demands and lower inertial forces transmitted to the floors.

- At the system level, the SMA dowel-type connections provide modestly larger displacement with near elimination of permanent deformation and reduced inertial demands due to lower floor acceleration. The results of the quantitative performance assessment showed that the SMA connections increase by 26% the peak interstorey drift but a 90% reduction in residual drift and 8% decrease in peak floor acceleration (compared to the steel connection results).

7.2 Recommendations for Future Work

The current study provides a numerical framework for evaluating the seismic performance of braced timber frames (BTFs) with SMA dowel-type connections, yet it has some limitations. The modelling framework presented can be used in similar numerical studies; however, it is recommended to perform full-size hysteresis connection-level models to calibrate the spring models used. In addition, the SMA connections for the building prototypes were designed based on non standardized material properties; therefore, the design of these connections might not meet adequate safety standards. Further standardization of NiTi SMA dowel-type fasteners is required to complement the current guidelines in Canadian standards.

The findings of this study suggest that the current height restrictions for braced-timber-frame structures in the National Building Code of Canada (NBCC) could be reviewed to consider limited-ductility and moderate-ductility classifications. In parallel, the NBCC and Engineering Design in Wood (CSA 086) committees should consider incorporating a connection-based ductility assessment, calibrated from experimental data and refined design equations, to more accurately assign a system's ductility category. Furthermore, the connection design provisions could be expanded to account for additional ductility indicators, such as the connection's slenderness ratio and the brittle-to-ductile strength ratio introduced in this study. For using these ratios, experimental and numerical studies are recommended to validate their applicability and to establish definitive limits for future code adoption.

The work presented here can further be advanced considering the following recommendations:

- **Expand Analysis to Moderate-Ductility Category:** Future research could expand the analysis to frames classified as moderate-ductility and employ parametric studies that relate required ductility demand to possible adjustments in NBCC height limits.

- **Validate Numerical Results with Large-Scale Testing:** Full-scale shake-table experiments on single- and multi-storey braced frames with SMA and conventional steel connections are recommended to benchmark and verify the numerical predictions.
- **Conduct Life-Cycle Cost Analysis and Embodied-Carbon Modelling:** Future studies can investigate the life-cycle cost assessments and embodied-carbon quantification to capture the broader economic and environmental implications of adopting SMA, beyond its structural benefits.

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APPENDIX A

A1. Connection-Level Models

```
from openseespy.opensees import *

# ANALYSIS CONFIGURATION
test('EnergyIncr', 1.0e-8, 300, 0)
algorithm('KrylovNewton')
system('UmfPack')
numberer('RCM')
constraints('Plain')

# CYCLE PARAMETERS
nsteps = 500

# LOAD CYCLE DATA
peaks1 = [0.14, 0.28, 0.56, 0.84, 1.12, 2.24, 2.24, 2.24, 4.48, 4.48, 4.48, 6.7, 6.7, 9.0, 9.0, 9.0, 11.2, 11.2, 11.2, 13.4]
peaks2 = [0.14, 0.28, 0.56, 0.84, 1.12, 2.24, 2.24, 2.24, 4.48, 4.48, 4.48, 6.7, 6.7, 9.0, 9.0, 9.0, 11.2, 11.2, 11.2, 13.4]

# CYCLE ANALYSIS
for i in range(len(peaks1)):
    for j in range(1, 3):
        dU = -1.0 * (-1.0)**j * peaks1[i] / nsteps
        integrator('DisplacementControl', 2, 1, dU)
        analysis('Static')
        analyze(nsteps)
    for j in range(1, 3):
        dU = 1.0 * (-1.0)**j * peaks2[i] / nsteps
        integrator('DisplacementControl', 2, 1, dU)
        analysis('Static')
        analyze(nsteps)

# MODEL CLEANUP
wipe()
```

A2. Brace-Level Models - Pushover Monotonic

```
from openseespy.opensees import *

# ANALYSIS CONFIGURATION
test('EnergyIncr', 1.0e-8, 300, 0)
algorithm('KrylovNewton')
system('UmfPack')
numberer('RCM')
constraints('Plain')
```

```

# LOAD MONOTONIC PARAMETERS
nsteps = 500
peaks1 = [18]

# LOAD APPLICATION
for i in range(len(peaks1)):
    for j in range(1, 3):
        dU = -1.0 * (-1.0)**j * peaks1[i] / nsteps
        integrator('DisplacementControl', 22, 1, dU)
        analysis('Static')
        analyze(nsteps)

# MODEL CLEANUP
wipe()

```

A3. Single-Storey Frame-Level Model - Pushover Monotonic

```

from openseespy.opensees import *

# IMPORT AND MODEL INITIALIZATION
model('basic', '-ndm', 2, '-ndf', 3)

# MATERIAL DEFINITION
uniaxialMaterial('DowelType', 1, Fi, Kp, Ru, c, beta, gamma, eta, Dy, alpha_p, alpha_u, alpha_r, '-piecewise', D1, F1, D2, F2, D3, F3, D4, F4)
uniaxialMaterial('DowelType', 2, Fi, Kp, Ru, c, beta, gamma, eta, Dy, alpha_p, alpha_u, alpha_r, '-exponential', K0, R1, F0, Dc, Kd)
uniaxialMaterial('Elastic', 3, 1e10)

# ZERO-LENGTH ELEMENT DEFINITION
element('zeroLength', 5110, 11, 110, '-mat', 1, 3, '-dir', 1, 2, '-orient', 4, 4, 0, -4, 4, 0)
element('zeroLength', 5221, 22, 221, '-mat', 2, 3, '-dir', 1, 2, '-orient', 4, 4, 0, -4, 4, 0)

# ANALYSIS CONFIGURATION
system('BandGeneral')
constraints('Transformation')
numberer('RCM')
test('NormDisplIncr', 1.0e-12, 10, 3)
algorithm('Newton')
integrator('LoadControl', 0.1)
analysis('Static')

# GRAVITY LOAD ANALYSIS
analyze(10)
loadConst('-time', 0.0)

# ANALYSIS CONFIGURATION
test('EnergyIncr', 1.0e-8, 300, 0)

```

```

algorithm('KrylovNewton')
system('UmfPack')
numberer('RCM')
constraints('Plain')

# MONOTONIC LOAD APPLICATION
nsteps = 500
peaks1 = [25]
for peak in peaks1:
    for j in range(1, 3):
        dU = -1.0 * (-1.0)**j * peak / nsteps
        integrator('DisplacementControl', 22, 1, dU)
        analysis('Static')
        analyze(nsteps)

# MODEL CLEANUP
wipe()

```

A4. Single-Storey Frame-Level Model - Pushover Cyclic

```

from openseespy.opensees import *
# MODEL INITIALIZATION
model('basic', '-ndm', 2, '-ndf', 3)

# MATERIAL DEFINITION
uniaxialMaterial('DowelType', 1, Fi, Kp, Ru, c, beta, gamma, eta, Dy, alpha_p, alpha_u, alpha_r, '-piecewise', D1, F1, D2, F2, D3, F3, D4, F4)
uniaxialMaterial('DowelType', 2, Fi, Kp, Ru, c, beta, gamma, eta, Dy, alpha_p, alpha_u, alpha_r, '-exponential', K0, R1, F0, Dc, Kd)
uniaxialMaterial('Elastic', 3, 1e10)

# ZERO-LENGTH ELEMENT DEFINITION
element('zeroLength', 5110, 11, 110, '-mat', 1, 3, '-dir', 1, 2, '-orient', 4, 4, 0, -4, 4, 0)
element('zeroLength', 5221, 22, 221, '-mat', 2, 3, '-dir', 1, 2, '-orient', 4, 4, 0, -4, 4, 0)

# ANALYSIS CONFIGURATION
system('BandGeneral')
constraints('Transformation')
numberer('RCM')
test('NormDisplIncr', 1.0e-12, 10, 3)
algorithm('Newton')
integrator('LoadControl', 0.1)
analysis('Static')

# GRAVITY LOAD ANALYSIS
analyze(10)
loadConst('-time', 0.0)

# CYCLIC ANALYSIS CONFIGURATION
test('EnergyIncr', 1.0e-8, 300, 0)
algorithm('KrylovNewton')

```

```

system('UmfPack')
numberer('RCM')
constraints('Plain')

# CYCLIC LOAD APPLICATION
nsteps = 500
peaks1 = [0.24, 0.48, 0.96, 1.43, 1.91, 3.82, 3.82, 3.82, 7.65, 7.65, 7.65, 11.5, 11.5, 11.5, 15.3, 15.3, 15.3, 19.0, 19.0, 19.0, 22.9, 22.9, 22.9]
peaks2 = [0.24, 0.48, 0.96, 1.43, 1.91, 3.82, 3.82, 3.82, 7.65, 7.65, 7.65, 11.5, 11.5, 11.5, 15.3, 15.3, 15.3, 19.0, 19.0, 19.0, 22.9, 22.9, 22.9]

for i in range(len(peaks1)):
    for j in range(1, 3):
        dU = -1.0 * (-1.0)**j * peaks1[i] / nsteps
        integrator('DisplacementControl', 22, 1, dU)
        analysis('Static')
        analyze(nsteps)
    for j in range(1, 3):
        dU = 1.0 * (-1.0)**j * peaks2[i] / nsteps
        integrator('DisplacementControl', 22, 1, dU)
        analysis('Static')
        analyze(nsteps)

# MODEL CLEANUP
wipe()

```

A5. Multi-Storey Frame-Level Model – Nonlinear Time History Analysis

```

from openseespy.opensees import *

# NRHA_FUNCTION_DEFINITION
def runNRHA(Dt, Tmax, Dc, tNode, bNode, log, pflag=0):
    import numpy as np
    if pflag > 0:
        print("Starting runNRHA")

    testType = "NormDisplIncr"
    tollnit = 1.0e-7
    iterInit = 100
    algorithmType = "KrylovNewton"

    test(testType, tollnit, iterInit)
    algorithm(algorithmType)
    integrator("Newmark", 0.5, 0.25)
    analysis("Transient")

    Nsteps = int(Tmax / Dt)
    cIndex = 0
    controlTime = 0.0

```

```

ok = 0
mdrft = 0.0
mflr = 0

h = []
mdrftX = []
maccelX = [0.0]
bdg_h = 0.0

for i in range(len(tNode)):
    top2 = nodeCoord(tNode[i], 2)
    bot2 = nodeCoord(bNode[i], 2)
    dist = top2 - bot2

    bdg_h += dist
    h.append(dist)
    mdrftX.append(0.0)
    maccelX.append(0.0)

    if dist == 0:
        print("WARNING: Zerolength found in drift check")

while cIndex == 0 and controlTime <= Tmax and ok == 0:
    ok = analyze(1, Dt)
    controlTime = getTime()

    if pflag > 1:
        print(f"Completed {controlTime} of {Tmax} seconds")

    if ok != 0:
        print(f"~~~ Failed at {controlTime}- Reduced timestep by half...")
        Dtt = 0.5 * Dt
        ok = analyze(1, Dtt)

    if ok != 0:
        print(f"~~~ Failed at {controlTime}- Reduced timestep by quarter...")
        Dtt = 0.25 * Dt
        ok = analyze(1, Dtt)

    if ok != 0:
        print(f"~~~ Failed at {controlTime}- Trying Broyden...")
        algorithm("Broyden", 8)
        ok = analyze(1, Dt)
        algorithm(algorithmType)

    if ok != 0:
        print(f"~~~ Failed at {controlTime}- Trying Newton with Initial Tangent...")
        algorithm("Newton", "-initial")
        ok = analyze(1, Dt)
        algorithm(algorithmType)

    if ok != 0:

```

```

    print(f"~~~ Failed at {controlTime}- Trying NewtonWithLineSearch...")
    algorithm("NewtonLineSearch", 0.8)
    ok = analyze(1, Dt)
    algorithm(algorithmType)

if ok != 0:
    print(f"~~~ Failed at {controlTime}- exit analysis...")
    cIndex = -1

base_accelX = nodeAccel(bNode[0], 1) / 9.81
maccelX[0] = max(maccelX[0], abs(base_accelX))

for i in range(len(tNode)):
    tNode_dispX = nodeDisp(tNode[i], 1)
    bNode_dispX = nodeDisp(bNode[i], 1)
    cHt = h[i]
    cdrftX = abs(tNode_dispX - bNode_dispX) / cHt * 100

    mdrftX[i] = max(mdrftX[i], cdrftX)
    mdrft = max(mdrft, cdrftX)
    if mdrft == cdrftX:
        mflr = i + 1

if mdrft >= Dc:
    cIndex = 1
    mdrft = Dc
    break

log.write(f"FinalState: {cIndex} at {controlTime:.3f} of {Tmax:.3f} seconds\n")
log.write(f"PeakStoreyDrift: {mdrft:.2f}% at {mflr}\n")

log.write("PeakStoreyDriftX: ")
for drft in mdrftX:
    log.write(f"{drft:.2f} ")
log.write("%\n")

log.write("PeakFloorAccelerationX: ")
for accel in maccelX:
    log.write(f"{accel:.2f} ")
log.write("g\n")

if pflag > 0:
    print(f"PeakStoreyDrift: {mdrft:.2f}% at {mflr}")

if cIndex == -1:
    print(f":::::: ANALYSIS FAILED TO CONVERGE at {controlTime} of {Tmax} :::::")
elif cIndex == 0:
    print("##### ANALYSIS COMPLETED SUCCESSFULLY #####")
elif cIndex == 1:
    print("===== LOCAL STRUCTURE COLLAPSE =====")

```

```

# NRHA_PREPARATION
print("Performing NRHA")
import ReadRecord
eqrec = "RSN"

dt, nPts = ReadRecord.ReadRecord(eqrec+'.at2', eqrec+'.txt')
EQnameX = f'{eqrec}.txt'
Dt = dt
Tmax = 2.0 * dt * nPts

dataDir = f"THA-Outputs-{phinge}-{eqrec}"
if not os.path.exists(dataDir):
    os.makedirs(dataDir)
if not os.path.exists(f"{dataDir}/log"):
    os.makedirs(f"{dataDir}/log")

g = 9.81 * 1000
sfX = 1.0 * g

# GRAVITY_ANALYSIS
constraints('Transformation')
numberer('RCM')
system('UmfPack', '-lvalueFact', 30)
test('EnergyIncr', 1e-6, 500)
algorithm('NewtonLineSearch')
nG = 100
integrator('LoadControl', 1.0 / nG)
analysis('Static')
analyze(nG)
loadConst('-time', 0.0)

# MODAL_ANALYSIS
numModes = 2
lambda_vals = eigen('-fullGenLapack', numModes)
if len(lambda_vals) == 0:
    lambda_vals = eigen('-symmBandLapack', numModes)
record()

omega = []
f = []
T = []

pi = math.acos(-1.0)
for lam in lambda_vals:
    omega_val = math.sqrt(lam)
    frequency = omega_val / (2 * pi)
    period = 2 * pi / omega_val

    omega.append(omega_val)
    f.append(frequency)
    T.append(period)

```

```

# DAMPING_SETUP
w1 = omega[0]
w2 = omega[1]
zeta = 0.03
a0 = zeta * 2.0 * w1 * w2 / (w1 + w2)
a1 = zeta * 2.0 / (w1 + w2)
rayleigh(a0, a1, 0.0, 0.0)

# EXCITATION_DEFINITION
tsTagX = 2
timeSeries('Path', tsTagX, '-dt', Dt, '-filePath', EQnameX, '-factor', sfX)
pTagX = 2
pattern('UniformExcitation', pTagX, 1, '-accel', tsTagX)

# ANALYSIS_OBJECTS_SETUP
constraints('Penalty', 1e14, 1e14)
numberer('RCM')
system('UmfPack', '-lvalueFact', 40)

# NRHA_EXECUTION
run = EQnameX
log_file = f"{dataDir}/log_{run}.txt"
log = open(log_file, "w")
Dc = 2.5
startRTime = time.time()

tNode = []
bNode = []
runNRHA(Dt, Tmax, Dc, tNode, bNode, log, pflag=1)

totalRTime = (time.time() - startRTime)
log.write(f"Total runtime was {totalRTime:.1f} seconds\n")
log.close()

```

APPENDIX B

B1. ESFM Seismic Loads – Three-Storey Prototype Building

Total Seismic Weight			
TOTAL, W 16459.2 kN			
Seismic Parameters			
Height	Hn, height =	12.0	m
Location	Ottawa, City Hall		
Foundation	Class C	Table 4.1.8.4.-A	
Site Design Spectrum, S(T) = Sa (T):			
S (0.2) = min (S(0.2), S(0.5) =		0.382	
S (0.5) =		0.227	
S (1.0) =		0.116	
S (2.0) =		0.052	
S (5.0) =		0.013	
Seismic Categories	IE S(0.2) =	0.382	Table 4.1.8.5.-B
	IE S(1.0) =	0.116	Table 4.1.8.5.-B
Importance	I _E =	1.0	Table 4.1.8.5., Normal Importance
Fund. Lateral Period	Ta = 0.025Hn =	0.30	Clause 4.1.8.11. 3) b)
Building	Timber Structures Designed and Detailed According to CSA 086		
	Braced Frames with Ductile Connections, SC3		
	Rd =	1.5	Table 4.1.8.9., Moderate Ductility
	Ro =	1.5	Table 4.1.8.9., Moderate Ductility
	S(0.2)/S(5.0) =	28.7	
Higher Mode	Mv (T<=0.5) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11
	Mv (T=1.0) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11
	Mv (T=2.0) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11
Overturning Moment Reduction Factor	J =	0.96	
	J (T<=0.5) for S(0.2)/S(5.0) =	0.960	Table 4.1.8.11
	J (T=1.0) for S(0.2)/S(5.0) =	0.871	Table 4.1.8.11
	S (Ta) Mv (Ta) =	0.330	
	S(0.2) Mv(0.2) =	0.382	
	S(0.5) Mv(0.5) =	0.227	
	S(1.0) Mv(1.0) =	0.116	
	S (2.0) Mv (2.0) =	0.052	

Design Base Shear - Equivalent Static Force Procedure

b) regular structures that are less than 60 m in height and have a fundamental lateral period, T_a , less than 2 s in each of two orthogonal directions as defined in Article 4.1.8.8

V, min	$V, \min = S(2.0) M_v I_E W / (R_d R_o) =$	382.6	kN
V, max	$V, \max =$	1862.9	kN
	$2/3 S(0.2) I_E W / (R_d R_o) =$	1862.9	kN
	$S(0.5) I_E W / (R_d R_o) =$	1660.6	kN
V	$V = S(T_a) M_v I_E W / (R_d R_o) =$	2416.5	kN
Design Seismic Base Shear, V =		1863	kN

B2. ESFM Seismic Loads – Six-Storey Prototype Building

Total Seismic Weight

TOTAL, W 34252.3 kN

Seismic Parameters

Height	Hn, height =	24.0	m
Location	Ottawa, City Hall		
Foundation	Class C	Table 4.1.8.4.-A	
Site Design Spectrum, S(T) = Sa (T):			
	S (0.2) = min (S(0.2), S(0.5) =	0.382	
	S (0.5) =	0.227	
	S (1.0) =	0.116	
	S (2.0) =	0.052	
	S (5.0) =	0.013	
Seismic Categories	IE S(0.2) =	0.382	Table 4.1.8.5.-B
	IE S(1.0) =	0.116	Table 4.1.8.5.-B
Importance	I _E =	1.0	Table 4.1.8.5., Normal Importance
Fund. Lateral Period	Ta = 0.025Hn =	0.60	Clause 4.1.8.11. 3) b)
Building	Timber Structures Designed and Detailed According to CSA 086		
	Braced Frames with Ductile Connections, SC3		
	Rd =	1.5	Table 4.1.8.9., Moderate Ductility
	Ro =	1.5	Table 4.1.8.9., Moderate Ductility
	S(0.2)/S(5.0) =	28.7	
Higher Mode	Mv (T<=0.5) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11
	Mv (T=1.0) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11
	Mv (T=2.0) for S(0.2)/S(5.0)	1.000	Table 4.1.8.11

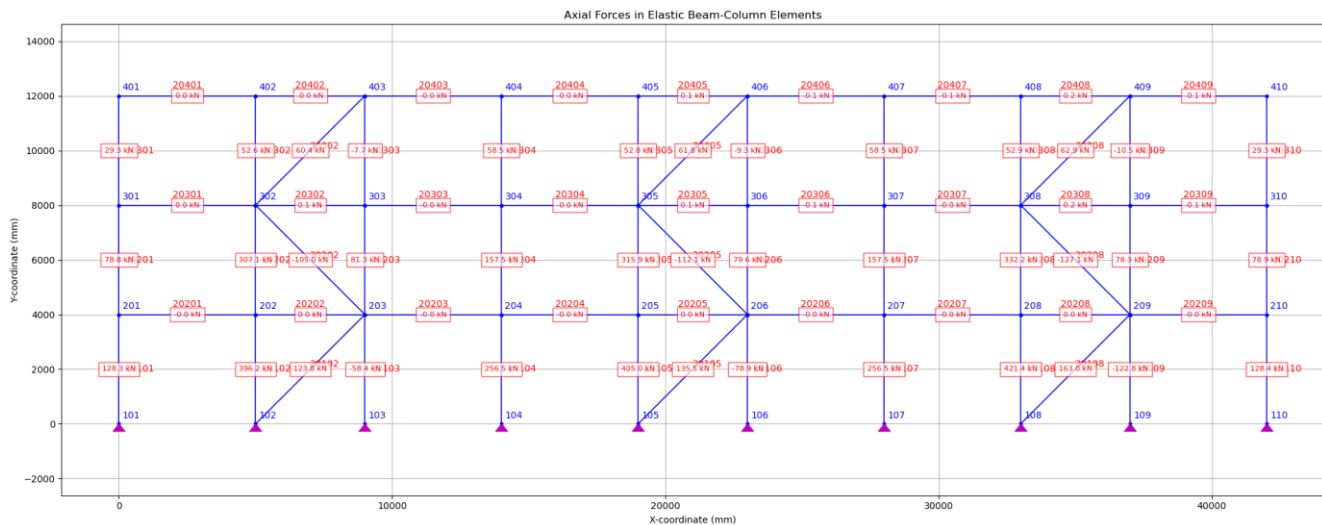
Overturning Moment Reduction Factor	J =	0.94	
	J (T<=0.5) for S(0.2)/S(5.0) =	0.960	Table 4.1.8.11
	J (T=1.0) for S(0.2)/S(5.0) =	0.871	Table 4.1.8.11
S (Ta) Mv (Ta) =		0.205	
S(0.2) Mv(0.2) =		0.382	
S(0.5) Mv(0.5) =		0.227	
S(1.0) Mv(1.0) =		0.116	
S (2.0) Mv (2.0) =		0.052	

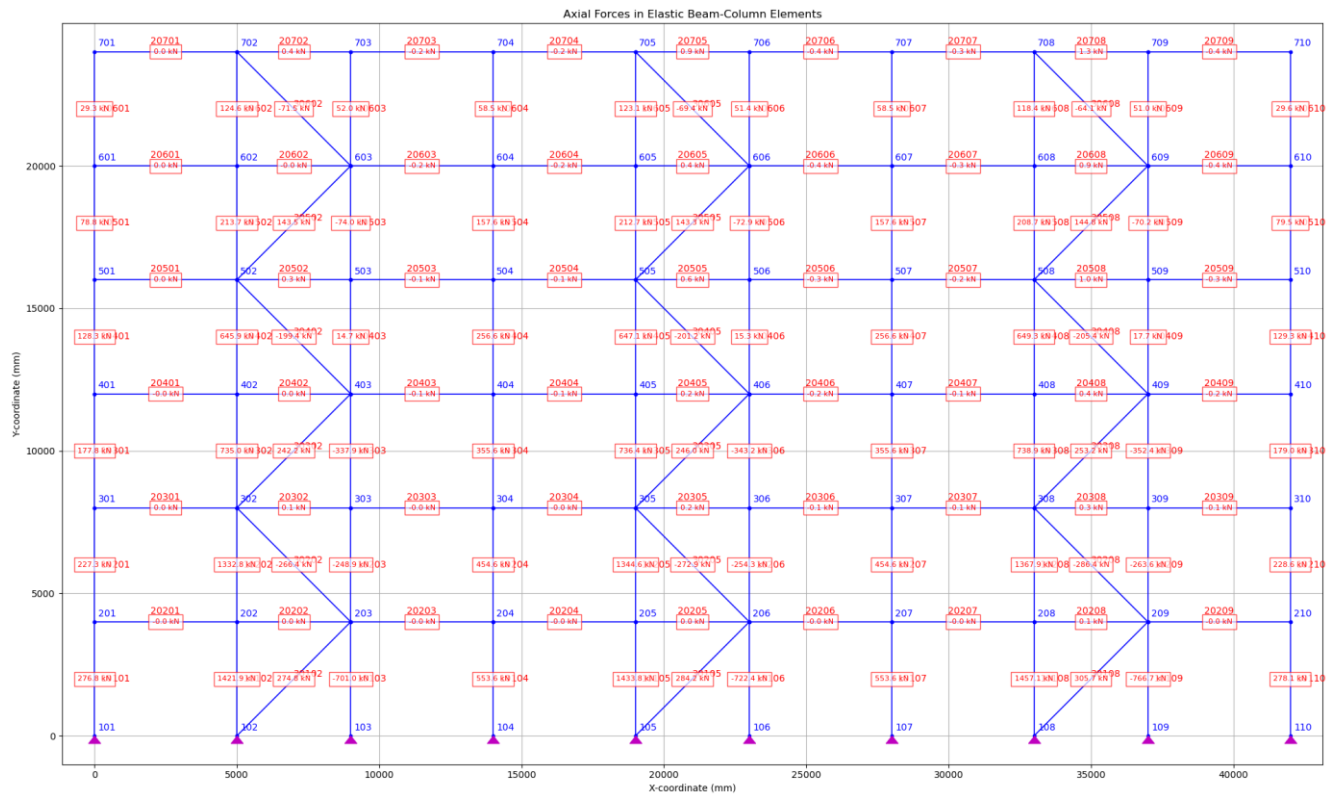
Design Base Shear - Equivalent Static Force Procedure

b) regular structures that are less than 60 m in height and have a fundamental lateral period, Ta, less than 2 s in each of two orthogonal directions as defined in Article 4.1.8.8

V, min	V, min = S(2.0) Mv IE W / (Rd Ro) =	796.2	kN
V, max	V, max =	3876.9	kN
	2/3 S(0.2) IE W / (Rd Ro) =	3876.9	kN
	S(0.5) IE W / (Rd Ro) =	3455.7	kN
V	V = S(Ta) Mv IE W / (Rd Ro) =	3117.7	kN
Design Seismic Base Shear, V = 1863 kN			

B3. Lateral Loads at Braces – Prototype Buildings





B4. Structural Design – Three-Storey Prototype Building

Column Level 1 to 4 (Ground Level – Storey 3)

Column Properties

Grade	SPF 12c-E
width, b =	215 mm
depth, d =	266 mm
length, L =	4000 mm
area, A =	57190 mm ²
volume, Z =	0.23 m ³
E =	9700 Mpa

Design

P_f =	428.9	kN
P_r =	653.8	kN

Floor Level Beams (Storey 1 and 2)

Beam Properties

Grade	SPF 20f-E
width, b =	215 mm
depth, d =	418 mm

Design

Bending

M_f =	106.8	kNm
M_r =	144.3	kNm

Shear

S =	6260943	mm ³	V _f =	85.4	kN
Ag =	89870	mm ²	V _r =	94.4	kN
Volume, Z =	0.45	m ³	W _f =	170.9	kN
I =	1308537157	mm ⁴	W _r =	289.5	kN
Deflection					
			Δ _{L/180} =	27.8	mm
			Δ _{Total} =	15.1	mm
			Δ _{L/360} =	13.9	mm
			Δ _{Live} =	6.5	mm

Roof Level Beam (Storey 3)

Beam Properties			Design		
Grade	SPF 20f-E		Bending		
width, b =	215	mm	M _f =	86.7	kNm
depth, d =	380	mm	M _r =	119.2	kNm
S =	5174333	mm ³	Shear		
Ag =	81700	mm ²	V _f =	69.4	kN
Volume, Z =	0.41	m ³	V _r =	85.8	kN
I =	983123333	mm ⁴	W _f =	138.8	kN
			W _r =	267.8	kN
Deflection					
			Δ _{L/180} =	27.8	mm
			Δ _{Total} =	15.1	mm
			Δ _{L/360} =	13.9	mm
			Δ _{Live} =	8.3	mm

Brace Level 1 (Ground Floor) with Steel-Bolted Connections: SC-3/8B-20-4/5

Connection Details				Member Properties		
d _F =	9.50	mm	bolt diameter	Grade	SPF 20f-EX	
n _R =	4		number of fastener rows	width, b =	215	mm
n _c =	5		number of bolts per row	depth, d =	342	mm
n _F =	20		number of bolts	length, L =	5650	mm
t _{plate} =	9.50	mm	plate thickness	G =	0.44	
a _L =	100	mm				
S _p =	65	mm				
S _q =	65	mm				
ep =	35	mm				
Design Criteria						

Strength:	Tf = 202 kN
	Tr = 219 kN
Overstrength Factor:	Brittle / Ductile = 2.2
	Ductile Resistance = 219 kN
	Brittle Resistance = 475 kN
Slenderness ratio:	t / dF = 18 >10 OK
	t = 166 mm
	dF = 10 mm

Brace Level 2 (Storey 1) with Steel-Bolted Connections: SC-3/8B-15-3/5

Connection Details			Member Properties		
d _F = 9.50 mm	bolt diameter		Grade	SPF 20f-EX	
n _R = 3	number of fastener rows		width, b = 215 mm		
n _c = 5	number of bolts per row		depth, d = 342 mm		
n _F = 15	number of bolts		length, L = 5650 mm		
t _{plate} = 9.50 mm	plate thickness		G = 0.44		
aL = 100 mm					
Sp = 65 mm					
Sq = 65 mm					
ep = 43 mm					

Design Criteria

Strength:	Tf = 158 kN
	Tr = 164 kN
Overstrength Factor:	Brittle / Ductile = 2.2
	Ductile Resistance = 164 kN
	Brittle Resistance = 356 kN
Slenderness ratio:	t / dF = 18 >10 OK
	t = 166 mm
	dF = 10 mm

Brace Level 3 (Storey 2) with Steel-Bolted Connections: SC-3/8B-9-3/3

Connection Details			Member Properties		
d _F = 9.50 mm	bolt diameter		SPF 20f-EX		
n _R = 3	number of fastener rows		Grade	EX	
n _c = 3	number of bolts per row		width, b = 175 mm		
n _F = 9	number of bolts		depth, d = 342 mm		
t _{plate} = 9.50 mm	plate thickness		length, L = 5650 mm		
aL = 100 mm			G = 0.44		
Sp = 70 mm					
Sq = 50 mm					
ep = 38 mm					

Design Criteria

Strength:	Tf = 78 kN
	Tr = 99 kN
Overstrength Factor:	Brittle / Ductile = 2.3
	Ductile Resistance = 99 kN
	Brittle Resistance = 230 kN
Slenderness ratio:	t / dF = 18 >10 OK
	t = 166 mm
	dF = 10 mm

Brace Level 1 (Ground Floor) with SMA-Bolted Connections: NC-3/8B-20-4/5

Connection Details			Member Properties	
d _F = 9.50 mm	bolt diameter		Grade	SPF 20f-EX
n _R = 4	number of fastener rows		width, b = 265 mm	
n _c = 5	number of bolts per row		depth, d = 342 mm	
n _F = 20	number of bolts		length, L = 5650 mm	
t _{plate} = 9.50 mm	plate thickness		G = 0.44	
aL = 100 mm				
Sp = 65 mm				
Sq = 65 mm				
ep = 35 mm				

Design Criteria

Strength:	Tf = 202 kN
	Tr = 248 kN
Overstrength Factor:	Brittle / Ductile = 1.9
	Ductile Resistance = 248 kN
	Brittle Resistance = 475 kN
Slenderness ratio:	t / dF = 18 >10 OK
	t = 166 mm
	dF = 10 mm

Brace Level 2 (Storey 2) with SMA-Bolted Connections: NC-3/8B-15-4/5

Connection Details			Member Properties	
d _F = 9.50 mm	bolt diameter		Grade	SPF 20f-EX
n _R = 3	number of fastener rows		width, b = 215 mm	
n _c = 5	number of bolts per row		depth, d = 342 mm	
n _F = 15	number of bolts		length, L = 5650 mm	
t _{plate} = 9.50 mm	plate thickness		G = 0.44	
aL = 100 mm				
Sp = 65 mm				
Sq = 65 mm				

ep = 43 mm

Design Criteria

Strength: Tf = 158 kN
Tr = 186 kN

Overstrength Factor: Brittle / Ductile = 1.9
Ductile Resistance = 186 kN
Brittle Resistance = 356 kN

Slenderness ratio: t / dF = 18 >10 OK
t = 166 mm
dF = 10 mm

Brace Level 3 (Storey 2) with SMA-Bolted Connections: NC-3/8B-9-3/3

Connection Details

dF = 9.50 mm bolt diameter
nR = 3 number of fastener rows
nC = 3 number of bolts per row
nF = 9 number of bolts
t_{plate} = 9.50 mm plate thickness
aL = 100 mm
Sp = 70 mm
Sq = 50 mm
ep = 38 mm

Member Properties

Grade SPF 20f-EX
width, b = 175 mm
depth, d = 342 mm
length, L = 5650 mm
G = 0.44

Design Criteria

Strength: Tf = 78 kN
Tr = 112 kN

Overstrength Factor: Brittle / Ductile = 2.1
Ductile Resistance = 112 kN
Brittle Resistance = 230 kN

Slenderness ratio: t / dF = 18 >10 OK
t = 166 mm
dF = 10 mm

B5. Structural Design – Six-Storey Prototype Building

Column Level 1 to 4 (Ground Level – Storey 3)

Column Properties			Compression Design		
Grade	SPF 12c-E		$P_f =$	759.6	kN
width, $b =$	315	mm	$P_r =$	1805.3	kN
depth, $d =$	380	mm			
length, $L =$	4000	mm			
area, $A =$	119700	mm ²			
volume, $Z =$	0.48	m ³			
$E =$	9700	Mpa			

Column Level 4 to 7 (Storey 3 – Storey 6)

Column Properties			Compression Design		
Grade	SPF 12c-E		$P_f =$	397.9	kN
width, $b =$	215	mm	$P_r =$	568.2	kN
depth, $d =$	228	mm			
length, $L =$	4000	mm			
area, $A =$	49020	mm ²			
volume, $Z =$	0.20	m ³			
$E =$	9700	Mpa			

Floor Level Beams (Storey 1 to 5)

Beam Properties			Design		
Grade	SPF 20f-E		Bending		
width, $b =$	215	mm	$M_f =$	106.8	kNm
depth, $d =$	418	mm	$M_r =$	144.3	kNm
$S =$	6260943	mm ³	Shear		
$A_g =$	89870	mm ²	$V_f =$	85.4	kN
Volume, $Z =$	0.45	m ³	$V_r =$	94.4	kN
$I =$	1308537157	mm ⁴	$W_f =$	170.9	kN
			$W_r =$	289.5	kN
			Deflection		
			$\Delta_{L/180} =$	27.8	mm
			$\Delta_{Total} =$	15.1	mm
			$\Delta_{L/360} =$	13.9	mm
			$\Delta_{Live} =$	6.5	mm

Roof Level Beam (Storey 6)					
Beam Properties			Design		
Grade	SPF 20f-E		Bending		
width, b =	215	mm	$M_f =$	86.7	kNm
depth, d =	380	mm	$M_r =$	119.2	kNm
S =	5174333	mm ³	Shear		
Ag =	81700	mm ²	$V_f =$	69.4	kN
Volume, Z =	0.41	m ³	$V_r =$	85.8	kN
I =	983123333	mm ⁴	$W_f =$	138.8	kN
			$W_r =$	267.8	kN
			Deflection		
			$\Delta_{L/180} =$	27.8	mm
			$\Delta_{Total} =$	15.1	mm
			$\Delta_{L/360} =$	13.9	mm
			$\Delta_{Live} =$	8.3	mm

Brace Level 1 and 2 (Ground Floor & Storey 1) with Steel-Bolted Connections: SC-1/2B-16-4/4

Connection Details			Member Properties		
$d_F =$	12.70	mm	Grade	SPF 20f-EX	
$n_R =$	4		width, b =	265	mm
$n_c =$	4		depth, d =	418	mm
$n_F =$	16		length, L =	5650	mm
$t_{plate} =$	12.70	mm	G =	0.44	
aL =	100	mm			
Sp =	100	mm			
Sq =	60	mm			
ep =	43	mm			
Design Criteria					
Strength:			$T_f =$	306	kN
			$T_r =$	308	kN
Overstrength Factor:			Brittle / Ductile =	2.3	
			Ductile Resistance =	308	kN
			Brittle Resistance =	713	kN
Slenderness ratio:			t / dF =	16	>10 OK
			t =	203	mm
			dF =	13	mm

Brace Level 3 and 4 (Storey 2 & 3) with Steel-Bolted Connections: SC-1/2B-15-3/5					
Connection Details			Member Properties		

$d_F =$	12.70	mm	bolt diameter	Grade	SPF 20f-EX
$n_R =$	3		number of fastener rows	width, $b =$	215 mm
$n_c =$	5		number of bolts per row	depth, $d =$	418 mm
$n_F =$	15		number of bolts	length, $L =$	5650 mm
$t_{plate} =$	12.70	mm	plate thickness	$G =$	0.44
$aL =$	100	mm			
$Sp =$	100	mm			
$Sq =$	60	mm			
$ep =$	48	mm			

Design Criteria

Strength:

$$T_f = 253 \text{ kN}$$

$$T_r = 289 \text{ kN}$$

Overstrength Factor:

$$\text{Brittle / Ductile} = 2.3$$

$$\text{Ductile Resistance} = 289 \text{ kN}$$

$$\text{Brittle Resistance} = 668 \text{ kN}$$

Slenderness ratio:

$$t / d_F = 16 > 10 \quad \text{OK}$$

$$t = 203 \text{ mm}$$

$$d_F = 13 \text{ mm}$$

Brace Level 5 and 6 (Storey 4 & 5) with Steel-Bolted Connections: SC-1/2B-9-3/3

Connection Details				Member Properties	
$d_F =$	12.70	mm	bolt diameter	SPF 20f-	
$n_R =$	3		number of fastener rows	Grade	EX
$n_c =$	3		number of bolts per row	width, $b =$	175 mm
$n_F =$	9		number of bolts	depth, $d =$	418 mm
$t_{plate} =$	12.70	mm	plate thickness	length, $L =$	5650 mm
$aL =$	100	mm		$G =$	0.44
$Sp =$	100	mm			
$Sq =$	60	mm			
$ep =$	28	mm			

Design Criteria

Strength:

$$T_f = 145 \text{ kN}$$

$$T_r = 173 \text{ kN}$$

Overstrength Factor:

$$\text{Brittle / Ductile} = 2.3$$

$$\text{Ductile Resistance} = 173 \text{ kN}$$

$$\text{Brittle Resistance} = 401 \text{ kN}$$

Slenderness ratio:

$$t / d_F = 16 > 10 \quad \text{OK}$$

$$t = 203 \text{ mm}$$

$$d_F = 13 \text{ mm}$$

Brace Level 1 and 2 (Ground Floor & Storey 1) with SMA-Bolted Connections: NC-1/2B-16-4/4

Connection Details				Member Properties		
$d_F =$	12.70	mm	bolt diameter	Grade	SPF 20f-EX	
$n_R =$	4		number of fastener rows	width, $b =$	265	mm
$n_c =$	4		number of bolts per row	depth, $d =$	418	mm
$n_F =$	16		number of bolts	length, $L =$	5650	mm
$t_{plate} =$	12.70	mm	plate thickness	$G =$	0.44	
$aL =$	100	mm				
$Sp =$	100	mm				
$Sq =$	60	mm				
$ep =$	43	mm				

Design Criteria

Strength:	$T_f =$	306	kN
	$T_r =$	349	kN
Overstrength Factor:	Brittle / Ductile =	2.0	
	Ductile Resistance =	349	kN
	Brittle Resistance =	713	kN
Slenderness ratio:	$t / d_F =$	16	>10 OK
	$t =$	203	mm
	$d_F =$	13	mm

Brace Level 3 and 4 (Storey 2 & 3) with SMA-Bolted Connections: NC-1/2B-15-3/5

Connection Details				Member Properties		
$d_F =$	12.70	mm	bolt diameter	Grade	SPF 20f-EX	
$n_R =$	3		number of fastener rows	width, $b =$	215	mm
$n_c =$	5		number of bolts per row	depth, $d =$	418	mm
$n_F =$	15		number of bolts	length, $L =$	5650	mm
$t_{plate} =$	12.70	mm	plate thickness	$G =$	0.44	
$aL =$	100	mm				
$Sp =$	100	mm				
$Sq =$	60	mm				
$ep =$	48	mm				

Design Criteria

Strength:	$T_f =$	253	kN
	$T_r =$	327	kN
Overstrength Factor:	Brittle / Ductile =	2.0	
	Ductile Resistance =	327	kN
	Brittle Resistance =	668	kN
Slenderness ratio:	$t / d_F =$	16	>10 OK
	$t =$	203	mm
	$d_F =$	13	mm

Brace Level 5 and 6 (Storey 4 & 5) with SMA-Bolted Connections: NC-1/2B-9-3/3

Connection Details			Member Properties		
$d_F =$	12.70	mm	bolt diameter	Grade	SPF 20f-EX
$n_R =$	3		number of fastener rows	width, $b =$	175 mm
$n_c =$	3		number of bolts per row	depth, $d =$	418 mm
$n_F =$	9		number of bolts	length, $L =$	5650 mm
$t_{plate} =$	12.70	mm	plate thickness	$G =$	0.44
$a_L =$	100	mm			
$S_p =$	100	mm			
$S_q =$	60	mm			
$e_p =$	28	mm			

Design Criteria			
Strength:	$T_f =$	145	kN
	$T_r =$	198.0	kN
Overstrength Factor:	Brittle / Ductile =	2.0	
	Ductile Resistance =	198	kN
	Brittle Resistance =	401	kN
Slenderness ratio:	$t / d_F =$	16	>10 OK
	$t =$	203	mm
	$d_F =$	13	mm

APPENDIX C

C1. Multi-Storey Brace Timber Frame Model (System-Level) – Nonlinear Time History Analysis

```
# IMPORTS & MODEL DEFINITION
from openseespy.opensees import *
model('basic', '-ndm', 2, '-ndf', 3)

# GEOMETRY DEFINITION & NODE CREATION
lev_height = 4000
b_btf_width = 4000
b_no_btf_width = 5000
btf_bays = [2, 5, 8]
btf_bays_str = [102, 105, 108]
n_bays = 9
n_storeys = 3 or 6
n_grid = n_bays + 1
n_levels = n_storeys + 1

heights = [i * lev_height for i in range(n_levels)]
grids = [0]
for bay in range(1, n_bays + 1):
    if bay in btf_bays:
        grids.append(grids[-1] + b_btf_width)
    else:
        grids.append(grids[-1] + b_no_btf_width)

node_coords = {}
for i, y in enumerate(heights, start=1):
    for j, x in enumerate(grids, start=1):
        node_id = int(f"{i:02d}{j:02d}")
        node(node_id, x, y)
        node_coords[node_id] = (x, y)

# BOUNDARY CONDITIONS
for j in range(1, n_grid + 1):
    base = int(f"01{j:02d}")
    fix(base, 1, 1, 0)

# DRIFT NODE GENERATION
def generateDriftNodes(n_grid, n_levels, interestgrid):
    if not (1 <= interestgrid <= n_grid):
        raise ValueError(f"interestgrid must be between 1 and {n_grid}")
    repGrid = interestgrid
    bNode = []
    tNode = []
    for level in range(1, n_levels):
        bottom_node = int(f"{level:02d}{repGrid:02d}")
        top_node = int(f"{(level+1):02d}{repGrid:02d}")
```

```

        bNode.append(bottom_node)
        tNode.append(top_node)
    return repGrid, bNode, tNode

# LOADS & MASSES
level_loads = {}

def trib_width(grid_index, grids):
    if grid_index == 1:
        return (grids[1]- grids[0]) / 2.0
    elif grid_index == len(grids):
        return (grids[-1]- grids[-2]) / 2.0
    else:
        left_bay = grids[grid_index- 1]- grids[grid_index- 2]
        right_bay = grids[grid_index] - grids[grid_index- 1]
        return 0.5 * (left_bay + right_bay)

# TIME SERIES & PATTERN DEFINITION
timeSeries('Linear', 1, '-factor', 1.0)
pattern('Plain', 1, 1)

# APPLY GRAVITATIONAL LOADS
for node_id in node_coords:
    level = node_id // 100
    grid = node_id % 100
    if level in level_loads:
        tw_mm = trib_width(grid, grids)
        tw_m = tw_mm / 1000.0
        applied_load = level_loads[level] * tw_m
        load(node_id, 0.0,-applied_load, 0.0)

# NODAL MASS ASSIGNMENT
g_mm = 9.81 * 1000
for node_id in node_coords:
    level = node_id // 100
    grid = node_id % 100
    if level in level_loads:
        tw_mm = trib_width(grid, grids)
        tw_m = tw_mm / 1000.0
        applied_load = level_loads[level] * tw_m
        mass_value = applied_load / g_mm
        mass(node_id, mass_value)

# GRAVITY STATIC ANALYSIS
wipeAnalysis()
constraints('Transformation')
numberer('RCM')
system('UmfPack', '-lvalueFact', 30)
test('EnergyIncr', 1e-6, 500)
algorithm('NewtonLineSearch')
nG = 100
integrator('LoadControl', 1.0 / nG)

```

```

analysis('Static')
analyze(nG)
loadConst('-time', 0.0)

# MODAL ANALYSIS
numModes = 4
lambdas = eigen(numModes)

# EARTHQUAKE SETUP & FILE HANDLING
logging.basicConfig(level=logging.INFO, format="%(asctime)s [%(levelname)s] %(message)s")
logger = logging.getLogger(__name__)
eqrec = "RSN"
at2_file = f"{eqrec}.at2"
txt_file = f"{eqrec}.txt"
g = 9.81 * 1000
sfX = 1.0 * g
if not os.path.exists(at2_file):
    logger.error(f"Required file '{at2_file}' was not found!")
    raise FileNotFoundError(f"File '{at2_file}' not found.")
dt, nPts = ReadRecord.ReadRecord(at2_file, txt_file)
EQnameX = txt_file
Dt = dt
Tmax = 2.0 * dt * nPts
logger.info(f"Loaded earthquake record '{eqrec}': dt = {dt:.4f} s, nPts = {nPts}, Tmax = {Tmax:.4f} s")

# OUTPUT DIRECTORIES
dataDir = f"{btf_name}-{eqrec}"
logDir = os.path.join(dataDir, "log")
for directory in [dataDir, logDir]:
    if not os.path.exists(directory):
        os.makedirs(directory)

# RAYLEIGH DAMPING
zeta_target = 0.03
omega1 = math.sqrt(lambdas[0])
omega2 = math.sqrt(lambdas[2])
delta_omega = omega2 - omega1
denom = (omega2**2 - omega1**2)
alpha_M = 2 * zeta_target * omega1 * omega2 * delta_omega / denom
beta_K = 2 * zeta_target * delta_omega / denom
rayleigh(alpha_M, 0.0, 0.0, beta_K)

# NONLINEAR RESPONSE HISTORY ANALYSIS FUNCTION
def runNRHA(Dt, Tmax, Dc, tNode, bNode, log, pflag=0):
    test("NormDisplncr", 1e-7, 100)
    algorithm("KrylovNewton")
    integrator("Newmark", 0.5, 0.25)
    analysis("Transient")
    Nsteps = int(Tmax / Dt)
    cIndex = 0
    controlTime = 0.0
    ok = 0

```

```

mdrft = 0.0
mflr = 0
h = []
mdrftX = []
maccelX = [0.0]
bdg_h = 0.0
for i in range(len(tNode)):
    top_coord = nodeCoord(tNode[i])
    bot_coord = nodeCoord(bNode[i])
    dist = top_coord[1] - bot_coord[1]
    bdg_h += dist
    h.append(dist)
    mdrftX.append(0.0)
    maccelX.append(0.0)
while cIndex == 0 and controlTime <= Tmax and ok == 0:
    ok = analyze(1, Dt)
    controlTime = getTime()
    if ok != 0:
        Dtt = 0.5 * Dt
        ok = analyze(1, Dtt)
    if ok != 0:
        Dtt = 0.25 * Dt
        ok = analyze(1, Dtt)
    if ok != 0:
        algorithm("Broyden", 8)
        ok = analyze(1, Dt)
        algorithm("KrylovNewton")
    if ok != 0:
        algorithm("Newton", "-initial")
        ok = analyze(1, Dt)
        algorithm("KrylovNewton")
    if ok != 0:
        algorithm("NewtonLineSearch", 0.8)
        ok = analyze(1, Dt)
        algorithm("KrylovNewton")
    if ok != 0:
        cIndex = -1
base_accelX = nodeAccel(bNode[0], 1) / 9.81
maccelX[0] = max(maccelX[0], abs(base_accelX))
for i in range(len(tNode)):
    t_disp = nodeDisp(tNode[i], 1)
    b_disp = nodeDisp(bNode[i], 1)
    cHt = h[i]
    cdrift = abs(t_disp - b_disp) / cHt * 100
    mdrftX[i] = max(mdrftX[i], cdrift)
    if mdrftX[i] == cdrift:
        mflr = i + 1
    mdrft = max(mdrft, cdrift)
if mdrft >= Dc:
    cIndex = 1
    break
log.write(f"FinalState: {cIndex} at {controlTime:.3f} s of {Tmax:.3f} s\n")

```

```

log.write(f"Peak Overall Storey Drift: {mdrft:.2f}% at Floor {mflr}\n")
log.write("Storey Peak Drifts:\n")
for i, drft in enumerate(mdrftX, start=1):
    log.write(f"  Storey {i}: {drft:.2f}% drift\n")
log.write("Peak Floor Acceleration X: " + " ".join(f"{acc:.2f}" for acc in maccelX) + " g\n")

# RUN NRHA
tsTagX = 2
timeSeries('Path', tsTagX, '-dt', Dt, '-filePath', EQnameX, '-factor', sfX)
pTagX = 2
pattern('UniformExcitation', pTagX, 1, '-accel', tsTagX)
constraints('Penalty', 1e14, 1e14)
numberer('RCM')
system('UmfPack', '-lvalueFact', 40)
run = EQnameX
log_file = f"{dataDir}/log_{run}.txt"
log = open(log_file, "w")
Dc = 2.5
startRTime = time.time()
interestgrid = 5
repGrid, bNode, tNode = generateDriftNodes(n_grid, n_levels, interestgrid)
totalRTime = time.time() - startRTime
log.write(f"Total runtime was {totalRTime:.1f} seconds\n")
log.close()

```