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SOFTWARE DEVELOPMENT FOR CAPACITY-BASED DESIGN OF TIMBER BRACED FRAMES: METHODOLOGY AND PERFORMANCE EVALUATION

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ABSTRACT: Timber structures are receiving increasing attention due to their environmental benefits, renewability, and alignment with low-carbon construction goals. Among timber-based seismic force resisting system (SFRS), the use of timber-braced frames (TBFs) is attracting growing interest owing to their structural simplicity, economic viability, and effectiveness in resisting seismic and wind loads. Despite these advantages, the widespread implementation of TBFs remains limited due to the lack of explicit design provisions and guidance within the Canadian Standard for Engineering Design in Wood (CSA O86), particularly with respect to ductility targets and capacity-based design methodologies. In addition, the absence of dedicated software and practical tools presents a challenge for structural designers in accurately analyzing and designing such systems.

This study proposes a comprehensive analysis and design methodology for TBFs, enabling engineers to meet key performance requirements, including specified ductility levels and the principles of capacity-based design through appropriate detailing. The methodology is implemented in a custom-developed software tool that facilitates the design process in accordance with the National Building Code of Canada (NBCC) and CSA O86. The software conducts Equivalent Static Force Procedure (ESFP) following NBCC provisions, employing Finite Element Method (FEM) principles. Connections at the braces' ends are considered the primary energy-dissipating components and the only source of nonlinearity, while capacity design principles are applied to all other structural members to ensure elastic behaviour under seismic loading. To validate the accuracy and applicability of the proposed methodology and developed software, a design example of a TBF with both limited and moderate ductility levels is presented. Nonlinear Time History Analyses (NLTHA) are conducted to evaluate the seismic performance. The results confirm that the frames designed using the developed software comply with seismic requirements and maintain structural integrity under the induced lateral loads.

KEYWORDS: Timber Braced Frame (TBF), Seismic Force Resisting System (SFRS), Capacity-based design Principles, Ductility, Nonlinear Time History Analysis (NLTHA).

1 – INTRODUCTION

In recent years, Canada has witnessed substantial growth in mass timber construction, motivated by the environmental benefits of wood as a sustainable building material with a low carbon footprint, alongside advancements in engineered wood products, ease and speed of construction, competitive costs, and active promotion of its structural applications [1, 2]. Among the primary SFRS in mass timber structures, such as Cross-Laminated Timber (CLT) shear walls and TBFs, TBFs represent one of the most efficient, straightforward, and cost-effective solutions for lateral load resistance in timber construction [3-5]. Unlike conventional steel-braced frames, which dissipate energy

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through the yielding or buckling of steel braces, the simplicity of TBFs lies in achieving ductility and energy dissipation through the yielding of their end brace connections. These connections are specifically detailed to exhibit ductile behaviour at predetermined locations, effectively functioning as structural fuses within the system [6, 7]. Typically, the ductility of connections is assessed through experimental testing, whereas system-level ductility is evaluated using inelastic analyses of the structure under lateral loading conditions [5]. Studies indicate that in TBFs ductility of the system can vary depending on the type of end brace connections. End brace connections utilizing slender bolts and timber rivets have demonstrated superior energy dissipation and ductility [8-10]. Although TBFs are included in the NBCC [11], CSA O86 [12] lacks explicit design provisions for achieving the targeted ductility levels and does not offer detailed guidance for the design and verification of TBFs

In addition, existing commercial structural analysis software currently lack a comprehensive framework for modeling, analyzing, and designing TBFs in accordance with timber engineering principles and design standards, particularly with respect to the nonlinear behaviour of end-brace connections. As a result, designers are required to make assumptions and adopt multi-step procedures to model these systems, introducing uncertainties that can make the analysis and design process both complex and potentially unreliable. This limitation significantly hinders the adoption of TBFs as the seismic force resisting system in mass timber structures. To address these challenges, this paper presents a methodology for the analysis and design of TBFs as SFRS.

Furthermore, to facilitate the practical implementation of the proposed methodology, a specialized software framework has been developed to streamline the workflow for researchers and structural engineers. This framework ensures compliance with CSA O86 [12] and aligns with the provisions of the NBCC [11]. By enhancing design reliability and streamlining the overall process, the software supports the broader adoption of TBFs as an effective SFRS in timber construction.

It should be mention that the current study is a continuation of earlier research at the University of Alberta focused on the development of a software tool for the analysis and capacity-based design of TBFs. The initial phase, presented in WTCE2023 [13], primarily addressed software development and the implementation of the proposed methodology and internal module configurations. Since 2023, the program has undergone several enhancements, most notably the optimization of the sections and the improvement of connection models. While this paper summarizes the key components of the capacity-based design approach and underlying assumptions, it places greater emphasis on methodological refinements and performance evaluation to demonstrate the applicability and effectiveness of the proposed methodology and the developed software.

2 – ANALYSIS AND DESIGN METHODOLOGY

To achieve the project objectives, the analysis process begins with ESFP. This is carried out through a systematic procedure involving the definition of the structural configuration and location, assignment of structural properties, and subsequent analysis of TBFs using ESFP principles. The preliminary data regarding load combinations, and spectral response acceleration (S_a) are derived from the requirements specified in NBCC 2020 [11]. In the subsequent stage of the analysis, the capacity-based design concept is integrated within the framework. In accordance with seismic design principles, ductility and energy dissipation in TBFs are required to be concentrated at the end brace connections, while all other structural members must remain within their elastic range [5-7, 9].

Therefore, following recommendations from previous studies on TBFs [14] and steel-braced frames [15, 16], the developed TBF analysis method is formulated based on several key considerations. In the model, columns and struts are treated as beam-column members, with struts modeled as two beam elements connected to the columns via end pin connections. Columns are assumed to be continuous and laterally braced at the strut locations to ensure stability under lateral loading. Section changes in columns are permitted only when the frame height exceeds the maximum available length of prefabricated timber columns. Braces are modeled as truss elements, representing their axial load-carrying behaviour without bending resistance. The lateral distribution of seismic forces (F_1 to F_n), calculated based on the base shear obtained from the ESFP, is shown in Figure 1(a). Based on these forces and using the direct stiffness method, all frame members are analyzed.

The design of the end brace connections constitutes the next step in implementing the capacity-based design concept. Once the connections are designed based on the axial forces in the braces, obtained by ESFP, their expected strength is determined. This expected strength is then assigned to the corresponding braces as the minimum force they must be capable of resisting. The entire frame is subsequently re-analyzed using the newly assigned brace

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forces , based on the capacity-based design concept, and moment equilibrium is applied to each column to calculate the redistributed lateral forces ($Ƒ_1'$ to $Ƒ_n'$). Using the principles of the method of joints and sections, the internal forces in all TBF members are recalculated. As a result, the updated lateral force distribution, denoted as $Ƒ_n'$ and shown in Figure 1(b), differs from the initial F_n obtained from the ESFP.

The design phase begins after the re-analysis step and the determination of newly calculated member forces. In the design stage, a user-defined overstrength factor can be applied to introduce a safety margin in the design of the timber elements and non-dissipative connections. The approach and steps for analyzing and designing the structural members of TBFs within the software are illustrated in Figure 2 as a flowchart, outlining the sequential procedures involved in the modeling, analysis, and design process.

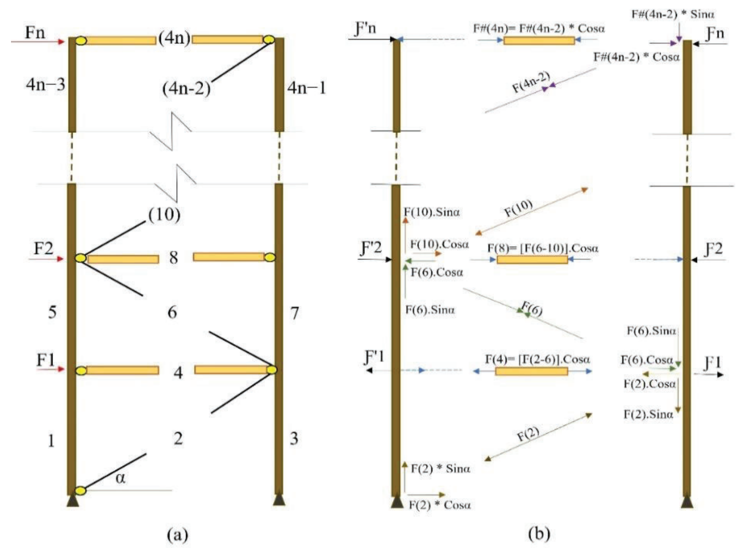

Figure 1. (a), Lateral distribution of seismic forces (Fn) from ESFP, (b) free-body diagram diagram of forces

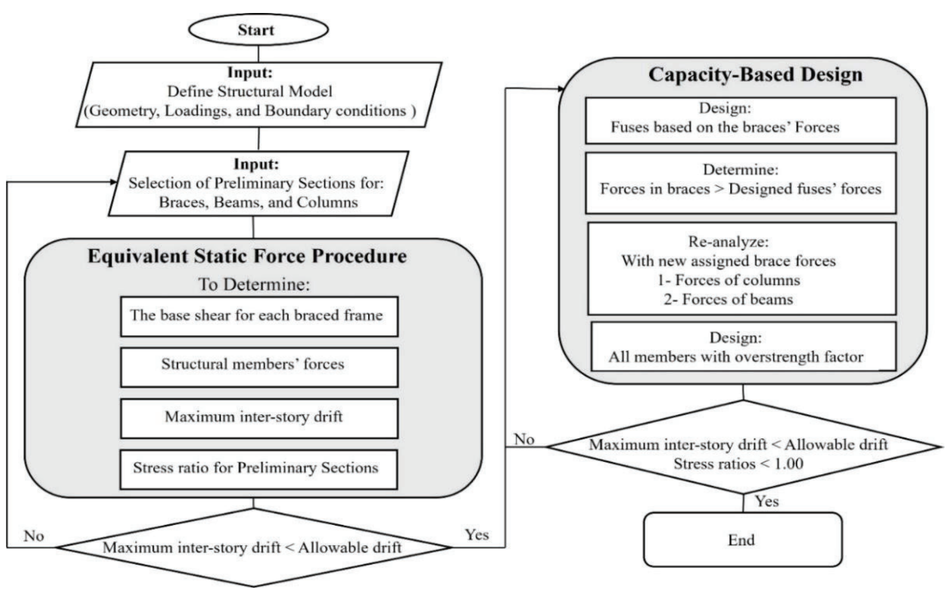

Figure 2. Flowchart of the Structural Modeling, Analysis, and Design Process

3- DEVELOPMENT OF SOFTWARE

The software developed in this study, implemented in Python [17], provides a unified platform that enables users to model, analyze, design, and optimize TBFs following the proposed methodology. Its interface integrates both general and specialized tab menus, guiding users through a structured, end-to-end workflow. The primary components of the software, including the structure information, loading, analysis, design, and results-saving sections are briefly described as follows:

3.1- PROJECT INFORMATION

The project information module serves as the initial step in the software workflow, enabling users to input essential project details such as model name, description, revision number, and personnel involved. The module automatically generates a file name and enforces standardized labeling to support efficient archetype management. Users then select the unit system, design, and loading codes (CSA O86-19 [12], NBCC 2015 [18]/2020 [11]), and identify the geographic location to retrieve relevant design load values for supported Canadian cities. The setup is completed by defining the building geometry, including the number of stories, bay dimensions, and brace locations.

3.2- LOADING SECTION

For vertical loading, the software allows users to define and extract distributed dead and live loads, as well as parameters required for snow load calculations based on the selected geographic location. The software calculates the contribution of each frame to vertical loads by considering the associated tributary areas. Two options are provided to users: including gravity loads in the analysis or limiting the analysis to lateral loads acting exclusively on the lateral-load resisting frame. When gravity loads are included, their effects are calculated for each frame and integrated into the analysis and design process, allowing for evaluation of structural performance based on project-specific requirements.

Additionally, the seismic weight of the building is calculated based on the defined vertical load inputs. Once the loads are specified, preliminary structural sections—comprising member dimensions and material properties—are assigned to the braces, beams, and columns. Material options include sawn lumber and glulam, with a range of sizes, species and grades available for user selection.

3.3- ANALYSIS SECTION

As previously noted, the analysis framework is based on the ESFP method, with user-defined options to include P-Δ effects and gravity loads. The framework computes both elastic and inelastic drifts and evaluates them against the drift limits prescribed in the NBCC [12], considering the structure's importance category. The analysis engine is based on the direct stiffness method, a standard approach in the Finite Element Method (FEM), and its results have been validated against SAP2000 [9] to ensure accuracy and reliability.

3.4- DESIGN SECTION

As previously stated, the primary objective of this study is to integrate a capacity-based design methodology into TBFs and support the practical implementation of this lateral load-resisting system in timber structures. The first stage involves the design of connections based on the internal forces obtained from ESFP, ensuring sufficient strength for reliable force transfer. The second stage addresses the design of structural members, which is governed by the expected strength of the connections to establish an appropriate strength hierarchy within the system. At this stage, the expected connection forces are assigned to the corresponding braces, followed by a re-analysis of the structure using moment equilibrium, along with the method of joints and method of sections, to account for the updated force demands. Following the force updates and completion of the re-analysis, the final stage of the process is divided into three parts—the design of braces, beams, and columns, following CSA O86-19 [12].

Upon completion of the structural member design, an optimization module is available to refine the selected sections. This module enables users to evaluate all suitable section alternatives based on stress ratios as the primary performance criterion. The software calculates the optimal section size for each member, and the finalized selections are compiled into a summary page for user review. At this stage, a re-run analysis is conducted to verify the structural integrity of the finalized sections, ensuring that drift values remain within allowable limits and that stress ratios meet design requirements. During

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this phase, the user retains the flexibility to revise section choices if needed, ensuring that all performance and code compliance criteria are fully satisfied before finalizing the sections.

3.5- IMPORT/EXPORT SECTION

As the final stage of the process, all results are exported and saved in a Python file format. This output can be utilized for generating structural drawings and preparing technical reports. Furthermore, the exported data serves as a foundation for advanced structural analyses, such as NLTHA, thereby extending the software's applicability to more sophisticated research and performance-based evaluation tasks.

4- DESIGN PERFORMANCE AND SOFTWARE EVALUATION

To evaluate the effectiveness of the proposed methodology for analyzing and designing TBFs, two structural models are developed using specialized structural analysis software. The final design outcomes from these models serve as the basis for further evaluation through NLTHA conducted in OpenSees [19]. The primary distinction between the two models lies in their assigned ductility levels: one is designed as a moderately ductile system, while the other is configured with limited ductility. The floor plan consists of three bays, each spanning four meters, and includes two braced frames in each principal direction. Both structural models represent three-storey residential buildings, with each storey having a height of three meters, resulting in a total building height of nine meters. This total height complies with the NBCC [11] height limitations, which permit a maximum of 15 meters for limited ductility systems and 20 meters for moderately ductile braced frames. The applied loads include a dead load of 0.75 kPa for the roof and 1.25 kPa for the floors. The live load is considered to be 2 kPa for the floors and 1.0 kPa for the partition walls. The designed sections obtained from the proposed methodology and developed software are presented in Table 1.

Assuming the structures are located in Vancouver, Canada, and subjected to crustal seismic events, the scenario-specific period range for such events is defined as 0.02 to 1.0 seconds. The corresponding moment magnitude–distance (Mw–R) scenario is characterized by a moment magnitude of 6.7 and a source-to-site distance of approximately 14 km [20]. Accordingly, a set of 11 ground motion records was selected and scaled in accordance with the NBCC 2015, Commentary J – Method A [18], to match the target response spectrum for Vancouver, considering Site Class D conditions and a 2% probability of exceedance in 50 years, shown in Figure 3. Table 2 presents the characteristics of the selected ground motion records along with their corresponding scaling factors.

4.1 END BRACE CONNECTIONS AND NONLINEAR BEHAVIOUR

Considering that the nonlinear behaviour of the system is assumed to be concentrated at the end-brace connections, while all other structural members are designed to remain elastic, the connection design in this study is directly based on experimental test data.

Table 1. Designed section properties

Moderately ductile			Limited ductility		
	Section Width*depth (mm)	Glulam Member Species/Grade		Section Width*depth (mm)	Glulam Member Species/Grade
All Columns	265*266	Spruce-pine / 20f-E	All Columns	265*266	Spruce-pine / 20f-E
Beams (Floor#1,2)	175*190	Spruce-pine / 20f-E	Beams (Floor#1,2)	175*190	Spruce-pine / 20f-E
Beams (Floor#3)	130*152	Spruce-pine / 20f-E	Beams (Floor#3)	130*152	Spruce-pine / 20f-E
Braces (Floor#1,2)	265*266	Spruce-pine / 20f-E	Braces (Floor#1)	315*304	Spruce-pine / 20f-E
Braces (Floor#3)	216*190	Spruce-pine / 20f-E	Braces (Floor#2,3)	265*266	Spruce-pine / 20f-E

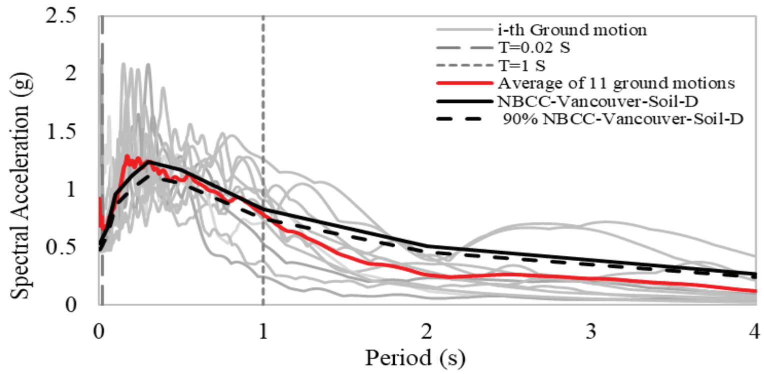

Figure 3. Comparison of scaled crustal ground motions with uniform hazard spectra for Vancouver

Table 2. Selected Ground Motion Records and Scaling Factors

Name	Earthquake	Magnitude (Mw)	R_{rup} (km)	V30 (m/sec)	Station Name	Scale Factor
V-01	Imperial Valley-02	6.95	6.1	213.4	El Centro Array #9	3.62
V-02	N. Palm Springs	6.06	4.4	345.0	North Palm Springs	1.14
V-03	Victoria Mexico	6.33	14.4	471.5	Cerro Prieto	2.41
V-04	Northridge-01	6.69	8.7	298.0	Arleta - Nordhoff Fire Sta	2.42
V-05	Northern Calif-03	6.50	27.0	219.3	Ferndale City Hall	3.27
V-06	Parkfield	6.19	9.6	289.6	Cholame - Shandon Array #5	3.23
V-07	San Fernando	6.61	22.6	450.3	Castaic - Old Ridge Route	4.33
V-08	Chi-Chi Taiwan	7.62	9.9	258.9	CHY101	1.87
V-09	Christchurch New Zealand	6.20	9.1	263.2	Papanui High School	2.30
V-10	Iwate Japan	6.90	31.1	248.2	Furukawa Osaki City	3.04
V-11	Kobe Japan	6.90	31.7	312.0	Tadoka	3.15

The existing dataset encompasses two main categories of fasteners: bolts and timber rivets. Further details regarding the characterization and performance of these connections are available from the previous research, which served as the first phase of the current study [13].

For NLTHA, the hysteretic response of the end-brace connections has been incorporated using the HystereticSM material model available in OpenSeesPy [19]. An optimization process using genetic algorithms was conducted to calibrate the parameters of the HystereticSM model against experimental data, ensuring an accurate representation of the cyclic behaviour of the connections. The optimized parameters for the current connection database have been integrated into the software framework, allowing users to apply them directly without the need for additional calibration.

A comparison between experimental test data for a bolted connection and the corresponding predictions from the optimized HystereticSM model is presented in Figure 4 (a). The sections designed by the developed software were used to create the OpenSees model. In this model, zero-length elements were employed to represent the nonlinear behaviour of the end brace connections, using the HystereticSM material model as previously described. The OpenSees model was developed based on the assumptions for the structural members outlined in Section 2 and is illustrated in Figure 4 (b).

4.2 NLTHA RESPONSES

The responses from the NLTHA are presented in Figures 5-7, which illustrate the maximum axial force, shear force, and bending moment demands in the columns; the maximum axial force in the braces; and the maximum axial force in the beams, each compared against the corresponding section capacity.

While axial force demand in the columns represents the most critical response parameter, the results indicate that all maximum demands remain within

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the design capacities of the respective members, except for the limited ductility frame subjected to the V-06 ground motion, where the column axial force exceeds its capacity by 2.3%. In contrast, shear and bending moment demands in the columns remain well within acceptable limits, not exceeding 30% of the respective section capacities. For the axial force demands in the braces of the moderate ductility frame, three ground motion records resulted in exceedances of the section capacity; however, in all cases, the exceedance was less than 10%. On average, the axial force demand across all 11 records remained within the capacity of the brace sections. For the limited ductility frame, all axial force demands in the braces remained within the capacity limits of the designed sections. Since the shear demands in all beams, for both the moderate and limited ductile frames, were less than 5% of their respective capacities, only the axial force responses are presented in Figure 7, indicating that axial force demands in all cases remain within the design capacities of the beam sections.

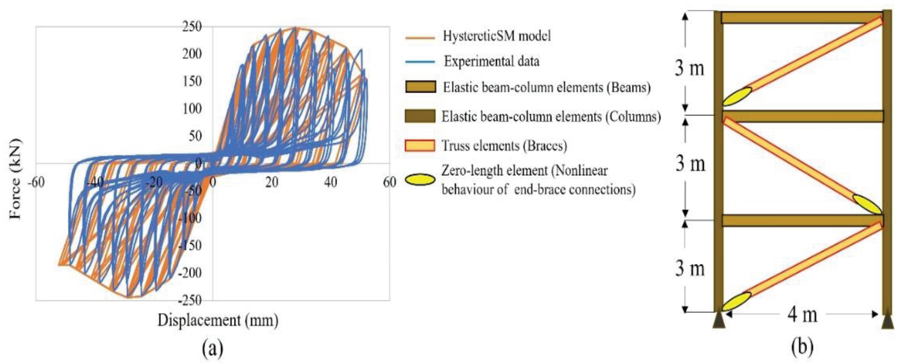

Figure 4. (a) Comparison of experimental data with the optimized HystereticSM model, (b) Schematic representation of the OpenSees model with the assigned member types and nonlinear end-brace connections

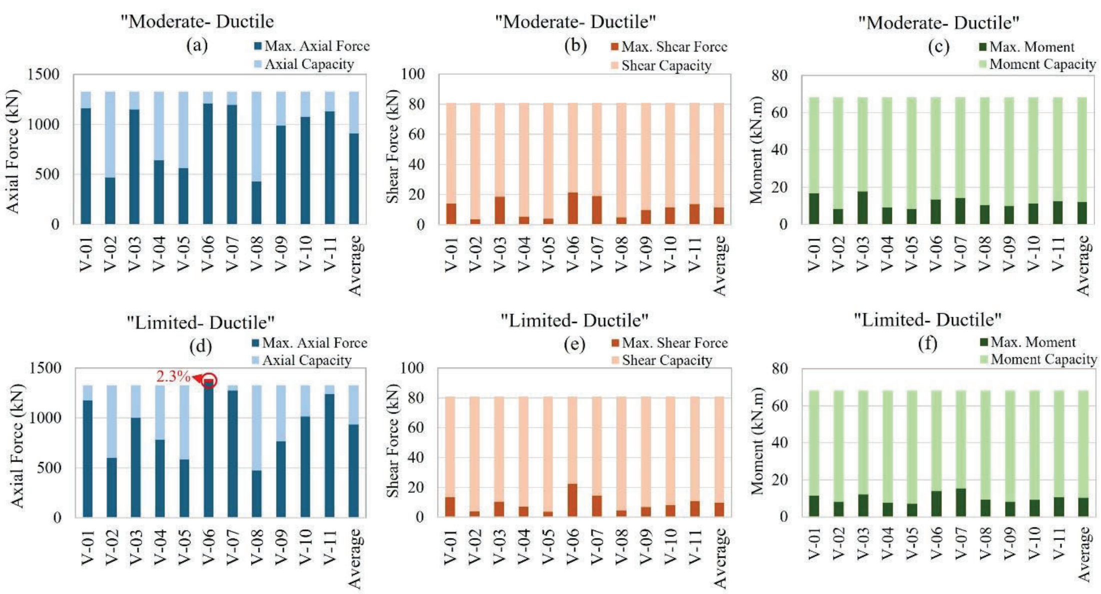

Figure 5. Maximum seismic responses of columns. For the moderately ductile frame: (a) axial force, (b) shear force, and (c) bending moment. For the limited ductility frame: (d) axial force, (e) shear force, and (f) bending moment.

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Figure 6. Maximum axial force of braces. For the moderately ductile frame: (a) floors 1 and 2, (b) floor 3. For the limited ductile frame: (c) floor 1, (d) floors 2,3.

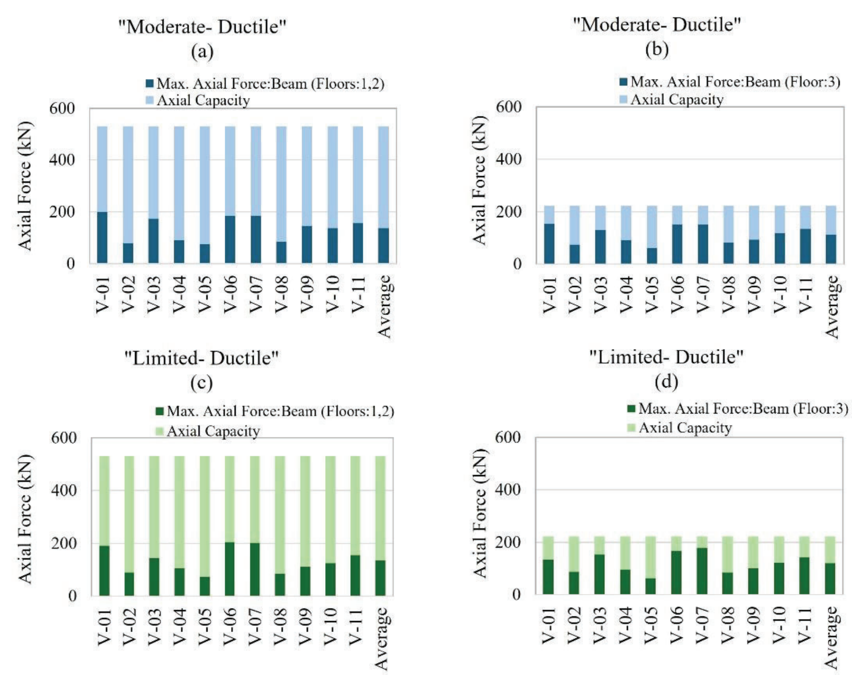

Figure 7. Maximum axial force of beams. For the moderately ductile frame: (a) floors 1 and 2, (b) floor 3. For the limited ductile frame: (c) floors 1,2, (d) floor 3.

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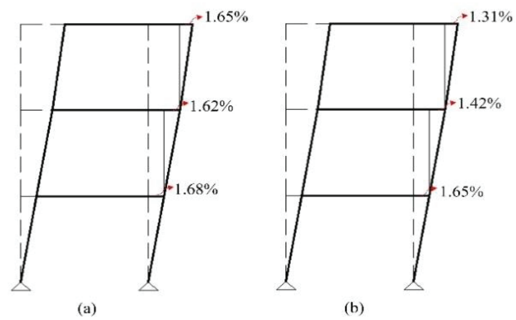

Figure 8. Maximum drift for: (a) Moderately ductile frame, (b) Limited ductile frame

As all inter-storey drift values remained within the allowable limit of 2.5% of the storey height, specified by the NBCC [18], for buildings in the normal importance category, only the maximum inter-storey drift response for the moderately ductile and limited ductile frames is presented in Figure 8, corresponding to the V-06 ground motion. Although the drift results demonstrate that the frames exhibit relatively high stiffness, this behaviour can be attributed to two primary factors. First, modifications to the column sections were restricted, resulting in a uniform cross-section along the full 9-meter column height. Second, engineered timber offers limited flexibility in fine-tuning section sizes compared to steel or concrete; in timber design, achieving the required strength typically necessitates relatively large increments in section dimensions. This response aligns with the material characteristics and design constraints inherent to timber structures and is considered acceptable within the context of timber engineering practice. However, further investigation is ongoing to optimize the proposed design methodology.

5 – CONCLUSION

Due to the lack of explicit design provisions in CSA O86 and limitations in practical tools for the design of TBFs, this study developed a comprehensive step-by-step methodology based on capacity-based design principles. The procedure follows the requirements of NBCC and CSA O86 from analysis through to final design and has been fully implemented in a custom-developed software platform. To evaluate the performance of TBFs designed using the proposed methodology, NLTHA was conducted. The results of the design procedure, implemented in the software, confirmed the seismic compliance and structural integrity of the three-storey TBF case study under seismic actions. The key outcomes of this research are summarized below:

1- The developed software streamlines the capacity-based design process for TBFs at the ESFP level in accordance with NBCC and CSA O86 requirements.

2- The generated design outputs can be directly used in NLTHA within OpenSeesPy, enabling further analysis for more complex structures or broader research objectives.

3- By covering a wide range of parameters and design preferences, the software can generate various structural configurations, supporting advanced studies such as the investigation of ductility factors in TBFs.

4- The nonlinear models created by the software can be applied to multiple research areas, including fragility analysis and machine learning-based prediction of seismic response or damage states in TBFs.

This work represents an important step towards closing the gap in TBF design tools within CSA O86, with ongoing developments planned to extend its capabilities, such as incorporating additional energy dissipation mechanisms to support future research and practical applications in seismic design of mass timber structures. More investigation is ongoing to verify the design methodology and the developed software.

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