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# THE ROLE OF INTERMODULAR CONNECTIONS IN THE GLOBAL BEHAVIOR OF HIGH-RISE MASS TIMBER BUILDINGS

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**ABSTRACT:** The use of mass timber as a pre-engineered product combined with prefabrication processes to assemble volumetric modular units contributes to optimizing the construction by reducing time, waste, and the weight of the structures for high-rise buildings located in high seismic regions. The performance of modular buildings subjected to lateral loads demonstrated that the connections between the modules play an important role in the diaphragm behavior and the structural response of the buildings. To study the building response, a parametric analysis was performed on 9-, 12-, and 18-story mass timber modular buildings simulated in OpenSees, to identify the influence of the stiffness of the intermodule connections and their location between the modular units. The analysis identified key parameters that govern the diaphragm behavior and how it affects the building's performance. The results demonstrated that the horizontal translational stiffness of the intermodular connections controls the diaphragm behavior thereby controlling the classification of the diaphragm as rigid, semi-rigid, and flexible diaphragm ranges for modular buildings. Additionally, the location of the intermodular connection modifies the diaphragm deflections and the envelope of the column force demands for the gravity system.

**KEYWORDS:** Mass timber, modular construction, intermodular connections, stiffness, high-rise buildings.

## 1 INTRODUCTION

### 1.1 INTRODUCTION TO MODULAR CONSTRUCTION AND MASS TIMBER

The architecture, engineering, and construction industry has acknowledged a shared commitment to increasing the speed of construction [1]. With the emergence of fabrication technology, including robotics, pre-fabrication of structural members and systems is possible. This results in the ability of structural frames or modules to be fabricated prior to arrival on site. Modular building construction takes advantage of this pre-fabrication ability and has been increasing in popularity to reduce the housing demands [2]. In most cases, these modular buildings use light steel frames to assemble modular units for residential buildings, and their dimensions are mainly limited by transportation requirements [3].

The benefits of modular buildings are reduction in construction waste, noise, and cost. Research has shown that 43% of the manufacturing waste can be recycled in a factory environment, resulting in waste of 5% of the total weight of the building compared to traditional construction with an average of 10 - 13% [4]. The use of modular units results in a 70% reduction in the number of trucks used to deliver construction materials to the site, which reduces the construction noise. Lastly, there is a 25 - 50% reduction in construction time of a mass timber building as compared to conventional building construction materials [4,5]. For all these reasons, mass timber modular buildings have the capability of increasing the speed of construction, reduction of waste, and reduction of noise during construction.

Modular building construction requires additional planning during the schematic design and design development phases to coordinate the floor plan such that it meets the functionality requirements of the building based on different arrangements of the modular units. Because of this additional planning, researchers have investigated optimal designs of the modular units for different occupancies [6,7]. Previous research shows that buildings with both office and residential occupancies can be developed using steel modular construction.

Connections are key elements that define the performance of modular buildings and modular units have three main types of connections [8]: intramodule, intermodule, and module to foundation connections. Intramodule connections connect the structural components within the module and are used to assemble the volumetric unit. These connections are designed to provide module stability during transportation and construction and to resist the gravity load demands. Intermodule connections constitute the module to module connection and transfer horizontal and vertical forces. These connections are designed for ease of construction and maintenance. Previous researchers have investigated the behavior of

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these types of connection in steel modular buildings and concluded that they can greatly influence the global behavior of the building [9,10]. The last type of connection is the module to foundation connection which transfers the vertical and lateral loads of the gravity system to the foundation. These connections are designed to restrain displacements and give support at the base of the building so each column can have the required conditions to develop its full strength. Concrete is mainly used for the foundation, and based on previous studies in steel modular buildings, these connections require additional on-site work due to welding and precautions to avoid corrosion [8].

Mass timber as a construction material is becoming more popular as a building construction material. From an analysis by Svatoš-Ražnjević et al. [11], the number of mass timber buildings and the number of stories in those buildings has increased between 2000 - 2021. Of the mass timber buildings constructed during this time, 7.3% of the buildings were classified as using volumetric units [11]. Fernandes Carvalho et al. [12] performed a case study analysis of nine modular mass timber buildings. These structures are used as schools, apartments, and hotels. All of them use either linear, planar, or volumetric modular units to assemble the mass timber structures. A hybrid high-rise hotel building was assembled using CLT panels, glulam beams and columns, and reinforced concrete walls up to 24 stories [13]. One of the major barriers to constructing more modular buildings using volumetric units is data on the behavior of inter and intramodular connections, how to simulate the building under lateral load demands, and proof-of-concept connection types, particularly for high-rise modular buildings.

### 1.2 PREVIOUS RESEARCH ON MODELING DIAPHRAGMS MASS TIMBER BUILDINGS

To simulate high-rise mass timber buildings under varying lateral force demands, researchers have used finite element (FE) modeling methodologies. However, previous researchers have utilized different methods of modeling mass timber diaphragm behavior.

Ávila et al. [14] simulated CLT diaphragm seismic behavior in a high-rise building with reinforced concrete shear walls. They concluded that using a rigid versus flexible diaphragm impacted the demands in the shear walls and the diaphragms themselves such that a rigid diaphragm produced larger demands in the walls than a flexible diaphragm; however, a rigid diaphragm produced smaller diaphragm demands than a flexible diaphragm [14]. Blomgren et al. [15] also studied CLT diaphragm behavior under seismic loads using the data from a two-story mass timber building tested at the University of California San Diego shake table. The conclusions of this research were that the ratio of the lateral stiffness of the CLT panels to the rigidity of the lateral system can influence whether the diaphragm behaves like a rigid, flexible, or semi-rigid diaphragm.
In general, diaphragms are designed as elastic elements so they can resist gravity loads and transfer the lateral loads without any substantial damage [16]. For code-based design, the diaphragm flexibility must be defined to simulate force distribution and to account for differential displacements of the diaphragm that may occur depending on its flexibility. For modular systems, diaphragms play an important role in the building behavior. During the construction stage, they provide stability to the module while as part of the building they transfer the force to the lateral system [17]. However, there is limited research on how to simulate modular building diaphragms, particularly considering the stiffness of the intermodular connections.

Diaphragms are classified based on their deformation capacity. Specifically, diaphragms are classified by the ratio between the maximum computed in-plane deflection of the diaphragm when subjected to lateral loads and the average drift of the elements of the seismic force-resisting system of the story below the diaphragm under consideration [18]. This ratio can be used to classify the diaphragm as rigid, semirigid, or flexible. The code-prescribed limit for rigid diaphragms is the ratio less than or equal to 0.5, flexible diaphragms greater than 2.0, and semirigid or stiff diaphragms between 0.5 and 2.0. While this diaphragm classification was developed for conventional construction, it has been applied to modular steel buildings in previous research of steel modular buildings [19] and therefore used by the authors.

Previous researchers have explored how intermodular connections can have an impact on the global behavior of a building. Chen et al. [20] developed and tested a steel intermodular connection that was later simulated by Peng et al. [21] to identify the behavior of steel modular buildings under seismic demands. They concluded that the fundamental period of the structure is 14% smaller when the intermodular connections are assumed to be rigid compared to the stiffness obtained from the tested steel connection that add more flexibility to the frame model.

Lacey et al. [22] and Fathieh [23] explored how the stiffness of the intermodule connections affected the structural response of modular buildings. Both researchers used individual spring elements to simulate the horizontal and vertical components of the connections and concluded that the translational stiffness of the intermodule connection had the biggest influence on the global structural behavior of the building. A connection with high translational stiffness resulted in a building with a smaller period and larger base shear forces, while a connection with smaller translational stiffness resulted in larger inter-story drifts. In addition, the translational stiffness of the intermodule connections has shown to increase the modal periods of the structure and cause higher mode participation [19]. The rotational stiffness of the intermodule connection showed to have little effect on the global building behavior [22,23].

This paper aims to explore how to simulate and design mass timber high-rise modular buildings by examining how the structural behavior was influenced by the stiffness of the intermodular connections, the


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intermodular connection location, the geometric transformation type, and the diaphragm property definition. Specifically, the objectives of this research are to: (1) develop a numerical modeling methodology to simulate mass timber high-rise modular buildings; (2) explore the influence of the stiffness of the intermodular connections on the diaphragm behavior of mass timber modular buildings; and (3) investigate the influence of intermodular connection and its location on the global structural response of mass timber modular buildings.

## 2 DESIGN DETAILS AND MODEL DESCRIPTION

### 2.1 BUILDING DESIGN

To achieve the research objectives, the authors used 9-, 12-, and 18-story modular buildings designed per to ASCE 7 [18] and IBC [24]. The buildings were assumed to be located in Seattle, Washington with site class D. Residential occupancy was considered to calculate the seismic design loads and a risk category II was assumed for the analysis. The gravity system of the structure used mass timber modular elements designed as volumetric units (Figure 1), and for the lateral force-resisting system double core reinforced concrete shear walls were located at the quarter points of the structure length (Figure 2). The building was designed with nine load combinations (Table 1), including seismic load cases. The loads considered are dead load (D), live load (L) due to residential occupancy and at the roof level (Lr), snow load (S), and earthquake load (Eh). The earthquake loads were calculated based on the equivalent lateral force method (ELF).

***Table 1:*** *Load combinations for Allowable Stress Design (ASD)*

| Comb | Load Combinations |
|---|---|
| 1 | D |
| 2 | D + L |
| 3 | D + Lr |
| 4 | D + S |
| 5 | D + 0.75L + 0.75Lr |
| 6 | D + 0.75L + 0.75S |
| 7 | $1.0D + 0.7E_h$ |
| 8 | $1.0D + 0.525E_h + 0.75L + 0.75S$ |
| 9 | $0.6D + 0.7E_h$ |

The gravity system was designed considering a corner-supported modular system, similar to previous research [4,7]. Each module consisted of four glue-laminated (glulam) columns, four perimeter glulam beams at the floor level, and four at the ceiling level. The floor was a cross-laminated timber (CLT) panel which spanned between the glulam beams and which was assumed to behave as a discrete diaphragm for each module. At the ceiling level, ponderosa pine CLT panels were used to ensure the stability of the module during the transportation and building construction. Since this panel did not resist gravity or lateral load demands, the authors chose a low grade CLT panel. Ponderosa pine is a biproduct of forest restoration for wildfire mitigation in the western states and exploring its usage within building construction would improve the sustainability of the building itself [25].

The superimposed dead load was 0.48 kN/m2 considering finishes at the floor levels. Live load was 2.40 kN/m2, which corresponds to a residential live load per IBC [24], 0.48 kN/m2 was applied at the ceiling level, and 0.96 kN/m2 for a non-occupied roof. The snow load for Seattle was 1.20 kN/m2 [26]. All the mass timber elements were Douglas-Fir and the dimensions were selected from commercially available material [27].

The frame elements used to assemble the modular units of the gravity system were designed per the National Design Specification (NDS) [28]. The centerline dimensions of the volumetric modules (Figure 1) were 3048 mm wide and 6096 mm long, with a total height of 4572 mm. For the 9- and 12-story buildings, the columns were 216 mm by 222 mm. The 18-story building used 260 mm square columns. The floor beams were 130 mm by 337 mm, while the ceiling beams were 130 mm by 298 mm. The floor beams and the ceiling beams were located such that there was a clear distance of 305 mm between the top of the ceiling and the bottom of the floor beam of the module above. This interstitial space was for plumbing and HVAC ductwork throughout the building and a common design for modular buildings. The CLT panel for the floor level was a 5-ply V2M1.1 grade 175 mm thick, and the ceiling panel was a 3-ply 105 mm thick CLT panel.

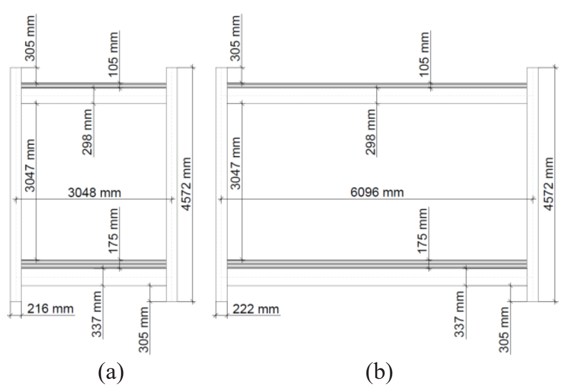


***Figure 1:*** *Module elevation a) in the short direction, b) in the long direction*

Using the designed mass timber modules, a floor plan was proposed based on a double-core shear wall system, similar to the Brock Commons Student Residence project [29]. The proposed floor plan and layout consisted of 41 modules per story, arranged in three rows in the E – W direction (Figure 2). The first and third rows contain 15 modules each while the second row contains 11 modules. At the quarter point along the length of the building, the concrete cores were located in a space equal to that of two modules (Figure 2). All the building cases have 457 mm thick concrete shear walls arranged in a C shape with an opening toward the center of the building. The wall thickness was selected to satisfy the drift limit requirements in both directions [18]. The modules are oriented with the longest side in the N – S direction, creating a space for the cores with a width-to-length ratio close to unity. The goal of the modular arrangement was to distribute the modules in a way to avoid structure irregularities or inherent torsion while still maintaining functionality of the building.

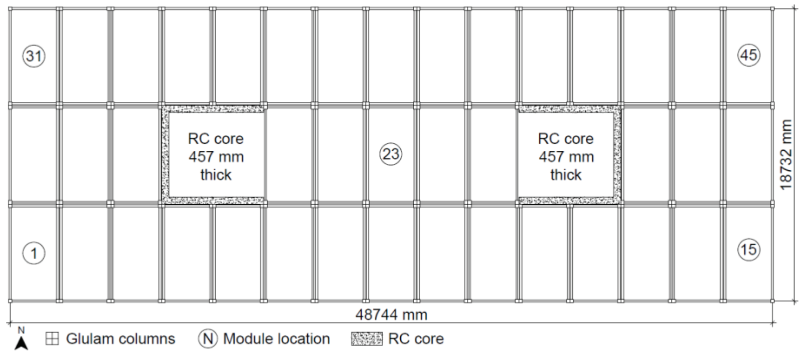


***Figure 2:*** *General plan configuration for modular buildings*

## 2.2 NUMERICAL MODELING

### 2.2.1 Modeling geometry

Three-dimensional (3D) finite element models were developed for the 9-, 12- and 18-story mass timber modular buildings using the open-sourced finite element program, OpenSees [30]. All of the mass timber elements were simulated using elastic beam-column elements for the beams and the columns. Zero-length elements were used at the base of the columns to simulate different boundary conditions and three elastic beam-column elements were used to simulate the interstitial space (Figure 3). The beams of the module used zero-length elements at the ends to provide a pinned boundary condition representing the intramodule connection. Finally, a rigid diaphragm property was assigned to the floor level of each module.

To connect the modules together throughout each floor, the intermodule connections were simulated using two-node link elements. This modeling methodology was similar to the modeling methodology implemented by previous research on steel modular buildings [7,22,31]. The two-node link spanned between the column nodes of each module on the same floor. To connect modules together between floors, a rigid zero-length element was used thereby assuming that load was always transferred vertically between the floors and continuity of the module columns was achieved through the connection. In addition, all intermodule connections remained elastic throughout the analysis. The benefit of considering link-type elements in the modeling process was the capacity to explore the impact of each DOF on the global behavior of the building. In addition, each DOF can be defined by assigning elastic materials that can be updated to include material nonlinearity.

Two locations for the intermodule connection were considered (Figure 3). The first location (Figure 3a) considers the intermodule connection (and two-node link) coinciding with the location of the floor beams. The second location (Figure 3b) considers the intermodule connection (and two-node link) located at the column joint. Both locations were considered to compare how the location of the intermodule connection influences the building behavior. For the remainder of the paper, the location shown in Figure 3a will be referred to as *the case with the link at the diaphragm level* and Figure 3b will be referred to as *the case with the link at the column joint location*.

The concrete shear wall elements were simulated using elastic beam-column elements with a fixed base. The dimensions of the walls depended on the location, for the E-W walls two elements were used for each side of the core, while in the N-S direction only one element was used. Connections between the modular elements and the concrete walls were modeled with two node link elements. Because the main objective of this study is to examine how the stiffness of the intermodular connections influences the global behavior of the building, the concrete walls were assumed to be undamaged and uncracked throughout the analysis. Therefore, only the elastic concrete material properties were utilized with an inertia reduction ratio of 0.70. For the purposes of this research, the intramodule connections were considered rigid enough to remain elastic and satisfy the gravity demands for the applied loads in each module

Second-order effects were included through corotational geometric transformation for all elements to take into account the influence of the tolerance limits of the modular elements during the construction process [32].

All the buildings were modeled using elastic uniaxial materials and manually adjusting the stiffness of the two-node link elements used to simulate the intermodule connections to perform the parametric analysis and evaluate the influence of each DOF of the connection on the modular building behavior. Glulam properties for elastic beam-column elements were used from commercial providers [27].


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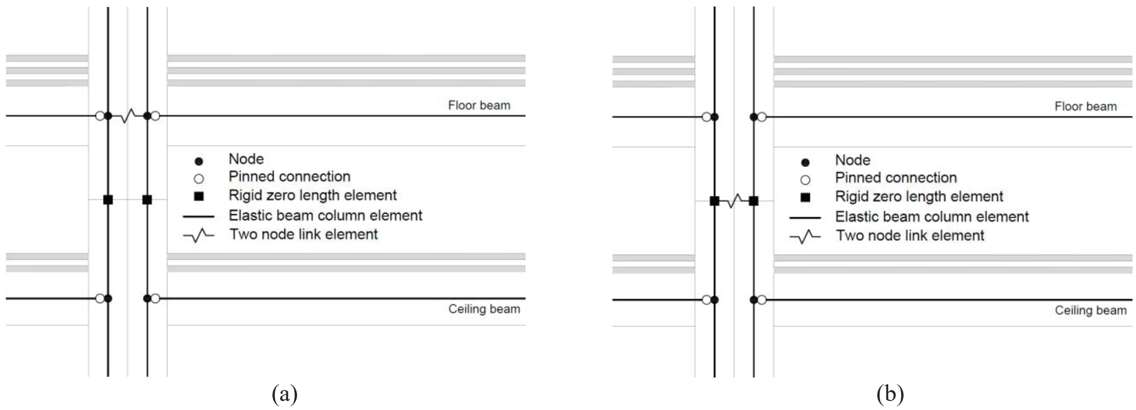



***Figure 3:*** *Horizontal link located a) at the diaphragm level, and b) at the column join level*

#### 2.2.2 Diaphragm modeling

The diaphragm of the case study buildings was assumed to be located at the floor level of the modules. The ceiling level had lower stiffness than the floor because ponderosa pine CLT was utilized for the ceiling panels. CLT diaphragms were considered to behave as rigid elements under design-level earthquake loads [33]. Therefore, the floor panels for each module were considered discrete elements with a rigid diaphragm property to avoid in-plane deformation. This modeling methodology also contributed to the analysis of the structural behavior of the entire floor diaphragm based on the stiffness of the intermodule connections.

The authors assumed a symmetric connection in the X and Y direction. With a parametric analysis the stiffness of the connections was modified in the translational and the rotational DOF to measure the diaphragm deflections. ASCE 7 [18] methodology was used to classify the building diaphragms according to their in-plane deflection compared to the lateral system deflection when both are subjected to lateral loading. The authors used the maximum deflection ratio to classify the diaphragm of each one of the simulations.

#### 2.2.3 Gravity and lateral loads

The gravity loads were included in the model as distributed loads along the module beams based on an equivalent tributary area at the floor and the ceiling beams. Self-weight was assigned as concentrated loads directly to the nodes for columns and the reinforced concrete shear walls. The structure effective mass was considered to be the total structure self-weight plus the superimposed dead load. The mass was assigned to the nodes at the floor and the ceiling level as concentrated mass. Lateral loads were assigned to the model based on the approaches used to simulate the diaphragm. When discrete diaphragms are assigned to each module, the lateral loads were uniformly distributed between the nodes of the columns located at the floor level. This approach allows to simulate the loading distribution required for semi-rigid or flexible diaphragms. In the case of a rigid diaphragm, the same uniform load distribution was applied.

### 2.3 SENSITIVITY ANALYSIS

A sensitivity analysis was performed to examine the influence of the intermodular connection location (Figure 3) and the influence of the stiffness of each DOF of the intermodular connection. This sensitivity analysis was also performed to increase computational efficiency. The results of the sensitivity analysis were carried through in the remainder of the investigation described in this paper. To perform this sensitivity analysis, a base model was defined. The base model consists of a building with rigid intermodular connections and a rigid diaphragm property assigned to each module in the building. The ranges of considered stiffnesses of the column links are summarized in Table 2.

***Table 2:*** *Parameters analyzed in the sensitivity analysis*

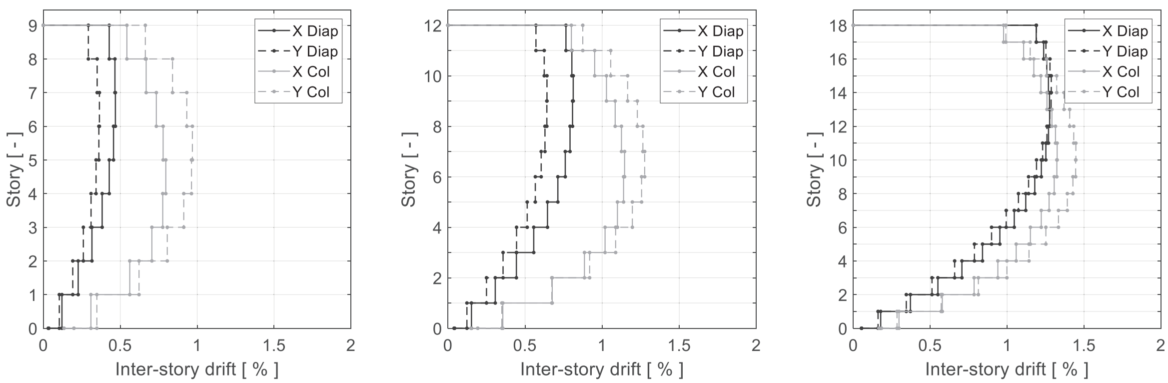
| Properties | Direction | Condition/Range |
|---|---|---|
| Link location | -- | Diaphragm or Column joint |
| Horizontal links | X, Y | Range: $1.75 \times 10^{-1}$ – $1.75 \times 10^{+8}$ kN/mm |
| Rotational links | θX, θY, θZ | Pinned or Fixed |

The ranges of stiffnesses provided in Table 2 were used to evaluate the influence of intermodule connection stiffness on diaphragm behavior, specifically the influence of the translational DOF stiffness. The horizontal translational stiffness (X and Y) was adjusted manually to fixed values while the remaining DOF of the connections were assumed to be rigid. The deformation ratio of the diaphragm, the modal periods, and the inter-story drifts of the buildings were calculated for each stiffness value.

## 3 RESULTS AND DISCUSSION

### 3.1 INTER-STORY DRIFTS

For all case study buildings and both locations of the intermodular connections (Figure 3), the maximum inter-story drift was below 1.5% (Figure 4). The inter-story drift increased when the intermodular connections were located at the column joint location. Analyzing each location independently, the buildings with connections at the diaphragm level, had higher inter-story drifts in the X direction than in the Y direction, while the buildings with connections at the column joint locations had larger drifts in the Y direction than in the X direction. The intermodule connection location also influenced the location of the maximum inter-story drift (Figure 4). For the buildings where the intermodular connections are located at the column joints, the maximum inter-story drift ratio was located at about 60% of the height of the building (Figure 4); whereas when the intermodular connection was located at the diaphragm, the maximum inter-story drift ratio was located at about 80% of the height of the building (Figure 4).



(a) (b) (c)

***Figure 4:*** *Design inter-story drifts for the diaphragm and column joint link cases: a) 9-story, b) 12-story, and c) 18-story*

### 3.2 DIAPHRAGM BEHAVIOR

Regardless of the location of the intermodule connection, as the translational DOF stiffness of the connection decreased, the diaphragm deflection increased exponentially (Figure 5). Based on static analysis results (Figure 5), the 9- and 12-story building required high translational stiffness for the intermodule connection to obtain the same diaphragm deflection ratio as the 18-story building with a less stiff connection. This horizontal stiffness of the intermodular connections influence the deflection of the diaphragms and requires a connection with larger translational stiffness to reach a certain deflection with shorter buildings. This response is opposite to the common behavior of the stiffness of traditional lateral force resisting systems (LFRS) that requires higher stiffness for taller buildings to satisfy the building drift demands.

The difference in diaphragm deflection based on the location of the intermodular connection is more pronounced for the 9-story building (Figure 5a), than for the 18-story building (Figure 5c) and for stiffer connections, than for more flexible connections. In general, buildings with intermodular connections at the column joint location require higher connection stiffness to obtain a rigid diaphragm property definition. On the contrary, regardless of the intermodular connection location, similar connection stiffnesses were required to obtain a semi-rigid or flexible diaphragm definition. Based on the diaphragm behavior shown in Figure 5, the authors assumed a rigid diaphragm could be obtained with an intermodular connection stiffness of 100 kN/mm, regardless of the building height as this connection stiffness results in diaphragm deflection less than 0.5.

The building in-plane configuration, particularly of the LFRS, was observed to have some influence over the diaphragm stiffness. All the cases were analyzed with the same building geometry, however, in the Y direction, when the lateral loads are applied on the long side of the building, the diaphragm deflection is greater than the deflection in the X direction for the same translational stiffness. This is because for the case study building, in the X direction, the lateral displacements are controlled by four shear walls, while in the Y direction, two shear walls control the displacements.

In addition to diaphragm deflection, the authors explored the impact that intermodular connection translational stiffness and location has on the model translational periods in each direction (Figure 6). The results (Figure 6) show that the translational periods are constant for connection stiffnesses greater than 100 kN/mm, which correspond to a rigid diaphragm (deflection ratio less or equal to 0.5). At connection stiffnesses less than 100 kN/mm, the building periods increase with small reductions in the translational stiffness of the intermodule connections. The location of the intermodule connection also has an impact on the translational period of the building. When the intermodule connection is located at the column joint, the buildings have a larger period than when the intermodule connection is located at the diaphragm. This result was observed regardless of building height.

Further analysis considered the modal shapes of the buildings to determine the direction of the translational modes and define if they are affected by the translational stiffness of the connections. Comparing the translational period in the X and Y direction, the same behavior occurred in all the story cases such that the buildings with the connections at the diaphragm change the direction of the fundamental period to the orthogonal direction in the semirigid range. For instance, in Figure 6 the influence of the location of the connection and its stiffness was important over the structure period. For the values at the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bottom of the curve the differences in the periods of the buildings were in some cases more than 1s while for the stiffer cases at the top, the differences remain in less than 0.5 s for all the buildings. For the column joint connection cases, the direction of the fundamental period remained the same for the entire range.

While Figure 5 and Figure 6 show the results of modifying the translational stiffness of the intermodule connections, Figure 7 shows the results of modifying the translational stiffness and the rotational DOF of the intermodule connections. The translational stiffness of the intermodule connections were varied in addition to the rotational DOF varying between pinned and fixed. This variation was performed in in both directions and with both locations of the intermodule connection. In the case of the fixed rotational DOF, the results shown in Figure 7 are the same as those shown in Figure 5 for comparison.

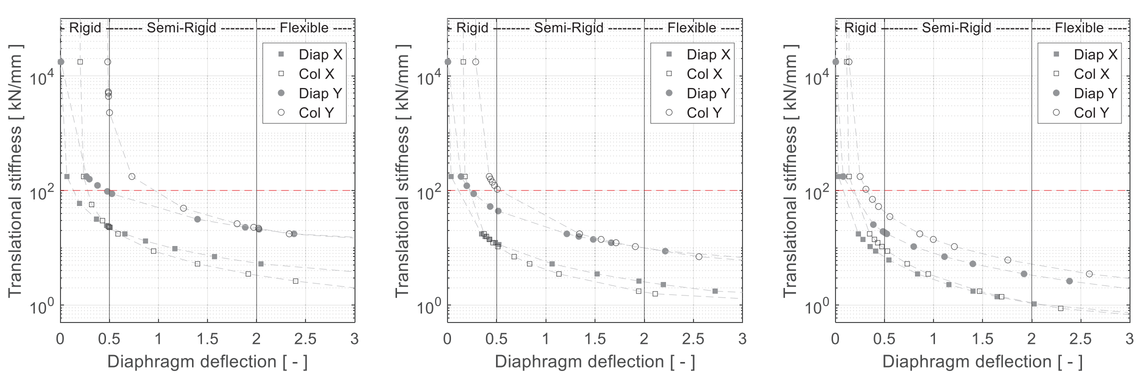
In the X and Y directions (Figure 7), there were small differences between the models with fixed and pinned rotational DOF when the connections were located at the diaphragm level (17.6% for the X direction and 13.2% for the Y direction, on average). However, when the intermodule connection is located at the column joint, the models with pinned rotational DOF had larger diaphragm deflections, on average 25.3% larger in the X direction and 29.8% larger in the Y-direction than models with fixed rotational DOF. In some cases, the location of the intermodular connection changed the diaphragm classification from rigid to semi-rigid. This effect was less pronounced for the 18-story building than for the 9-story building.

The simulation results in Figures 5 – 7 demonstrate that the translational stiffness of the intermodular connections had more of an influence on the diaphragm behavior than the rotational stiffness. This conclusion was consistent with previous research on modular buildings [22]

### 3.3 ELEMENT FORCES

The modular building design was implemented using an envelope of forces within the structural elements for the two intermodular connection locations (Figure 3). The envelopes of the design forces are shown in Figure 8 for the 12-story building. The forces in the other two buildings were proportional to their height.

The location of the intermodular connection did not substantially influence the axial force demands in the gravity columns (Figure 8a). Rather, there was an 8.7% difference between the axial force demands when the intermodular connections were located at the diaphragm as opposed to the column joint. However, the intermodular connection location did influence the shear demands within the gravity columns (Figure 8b and c). The shear force envelopes in the connection region for when the intermodular connection was located at the diaphragm was on average 33.6% lower in the strong axis (Figure 8b) and 66.8% lower in the weak axis (Figure 8c) than when the intermodular connection is located at the column joint. Similarly, the moment demands in the gravity columns were on average 13.2% lower in the weak axis (Figure 8d) and 26.4% lower in the strong axis (Figure 8e) when the intermodular connection was located at the diaphragm than at the column joint. Columns strong axis is located parallel to E-W direction (Figure 2).




(a) (b) (c)

*__Figure 5:__ Diaphragm flexibility for the diaphragm and column joint link cases: a) 9-story, b) 12-story, and c) 18-story*

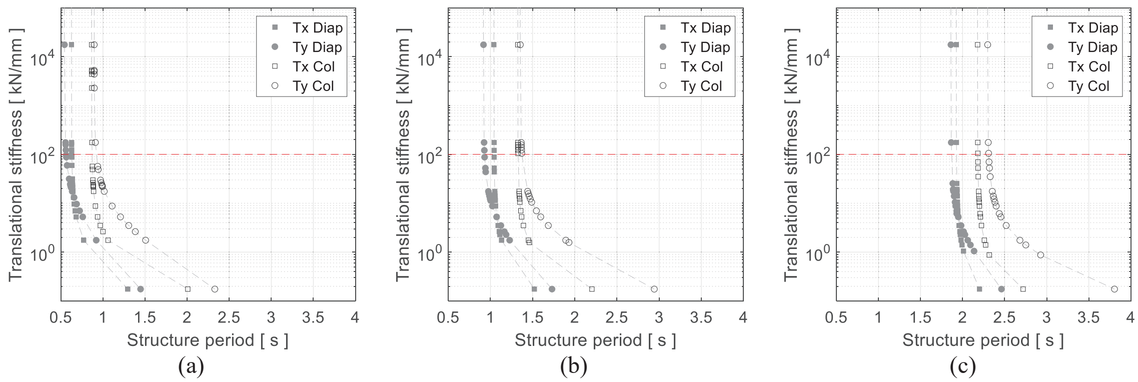



***Figure 6:** Structure period for the diaphragm and column joint link cases: a) 9-story, b) 12-story, and c) 18-story*

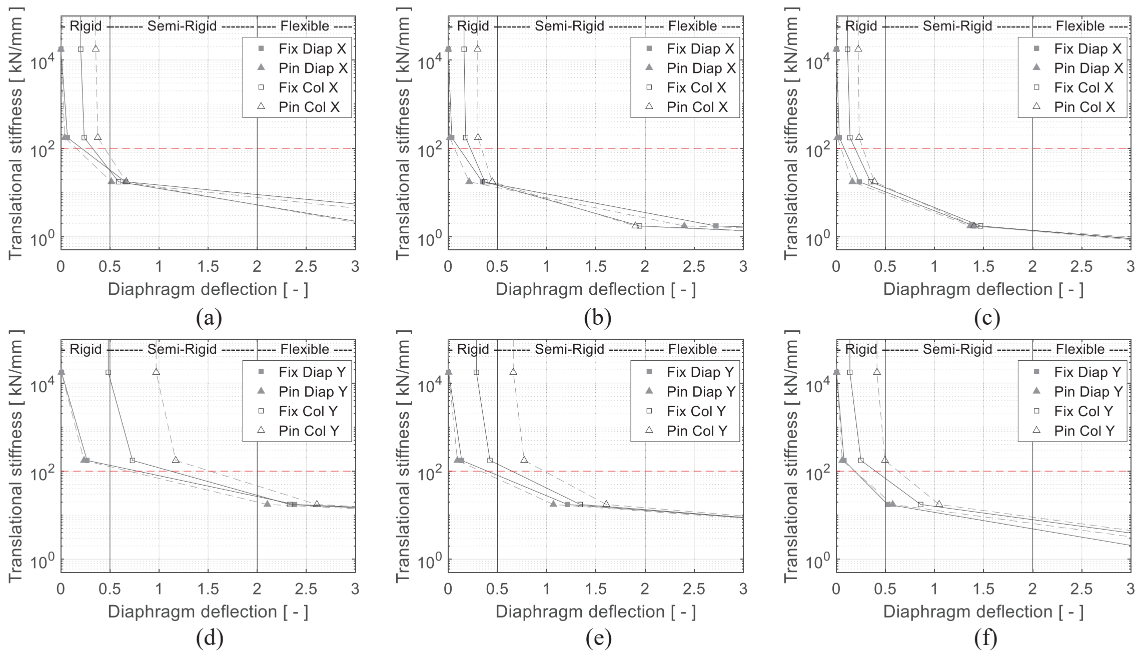



***Figure 7:** Diaphragm deflections for varying locations of intermodular connections with pinned and fixed rotational stiffness: a) 9-story X direction, b) 12-story X direction, c) 18-story X direction, d) 9-story Y direction, e) 12-story Y direction, and f) 18-story Y direction*

## 4 CONCLUSIONS AND FUTURE WORK

This paper presented the results of the analysis of a 9–, 12-, and 18-story mass timber modular building with two reinforced concrete cores. 3D elastic models were developed in OpenSees and used to evaluate the influence of the stiffness and location of intermodule connections on the global response of modular buildings under combined gravity and lateral loads. The modeling methodology developed in this research and summarized within this paper can be utilized to simulate high rise mass timber modular buildings. The numerical simulation approach with beam and column elements to assemble the volumetric units, generated similar response for mass timber modular buildings compared to steel modular buildings reported in previous studies. Therefore, the authors can conclude that the structural behavior described in this paper and the conclusions made herein can be generalized for modular construction independent to the material used to simulate the modular units.

The stiffness of the translational DOF of the intermodular connections showed an inverse exponential relation with the diaphragm deflections for all the building heights. However, taller buildings required a less stiff connection than shorter buildings to have a rigid diaphragm behavior. Figure 5 can be utilized by designers to determine the required connection stiffness to obtain specific diaphragm flexibility given a building height. Simulations demonstrated that the period of mass timber modular buildings remains constant if the diaphragm can be classified as rigid (Figure 6) regardless of the building height, however the building period increases with small reductions of the translational stiffness of the intermodule connections when the diaphragm is classified as either semi-rigid or flexible.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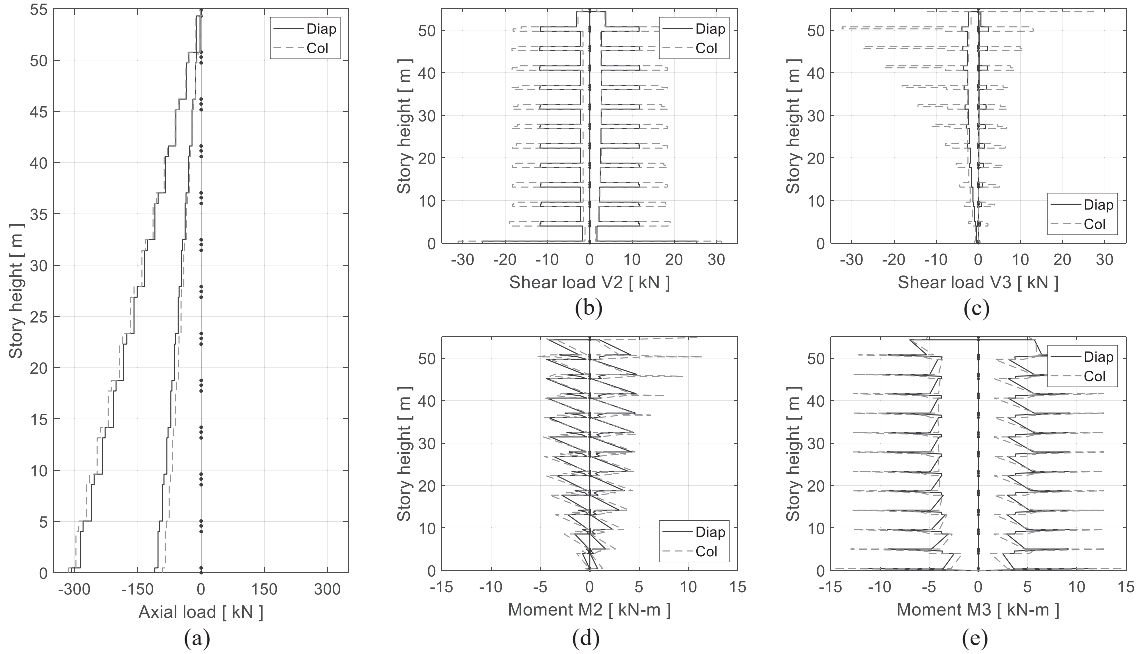


***Figure 8:*** *Design element forces for 12-story buildings: a) axial load, b) shear load in the strong axis, c) shear load in the weak axis, d) moment around the weak axis, e) moment around the strong axis*

The rotational DOF of the intermodule connections has little effect on the diaphragm deflections compared to the influence of the translational stiffness. When the location of the connections was considered, the rotational DOF of the intermodule connections showed a slight increase of the diaphragm deflections for the buildings with the connections at the column joint.

Two different locations of the intermodule connections for the modular buildings were explored in the analyses. The building performed better when the connection was located at the diaphragm level. Specifically, the periods and drifts distribution were influenced by the location of the connection. When the intermodule connection was located at the column joint, the connection required larger translational stiffness to obtain lower diaphragm deflections in all the buildings. In addition, when the intermodule connection was located at the column joint, the shear and moment demands in the column elements were higher.

To identify a better relation between the diaphragm flexibility and the in-plane configuration of the building, additional analyses should be performed to compare different model geometries, however, it is considered out of the scope of the current research. Future research on this topic should also explore the dynamic response of the structures to verify the building behavior under earthquake hazards and define the required strength to design and test the inter-modular connections.

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