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Fragility-based robustness assessment of steel modular building systems: Connection and building height

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ABSTRACT

Robustness is an important factor in determining structures' ability to withstand accidental extreme events. However, assessment of structural safety typically does not take probabilistic factors into account, which results in disregarding uncertainties even when extreme conditions are considered. The are limited studies in the literature, highlighting the need for fragility risk assessment of the impact of inter-module connections (IMCs) and building height of steel modular building systems (MBSs) subjected to progressive collapse scenario.

This paper investigates the robustness of steel modular building systems (MBSs) under progressive collapse scenarios that vary in connection type and building height. A nonlinear static pushdown analysis was carried out on 5-, 10-, and 15-story MBSs with bolted and post-tensioned rod IMCs, focusing on column removal during the analysis using OpenSees software. Results showed that taller structures are more robust due to their increased redundancy while they exhibit greater resistance to collapse than lower structures. Fragility analysis can be utilised to predict the probability of progressive collapse in the case of local damage. With the derived fragility functions, the probability of progressive collapse is quantified for different IMCs and building heights. By optimising connection types and building configurations, the results provide new insights into designing safer modular steel buildings.

1. Introduction

As part of the modern methods of construction (MMC), modular building system (MBS) involves prefabricating a building as a three-dimensional (3D) volumetric unit in a factory and assembling it on site to form a complete structure or major part of a structure [1]. Due to the numerous advantages of modular construction, such as quicker construction, less environmental impact, enhanced quality control, and improved workplace safety [2–5], MBS is a promising alternative to traditional framed structures. Typically, square hollow sections (SHS) and wide flange (W or UB) are used for columns and beams, respectively [6]. In recent research on inter-module connections (IMCs), the connections between modular volumetrics (boxed containers), and the performance of multi-story modular buildings under earthquake actions [7–16], wind actions [17–20], and abnormal loading circumstances - defined as extreme, non-standard scenarios such as blasts or impacts that differ from typical design loads and may lead to progressive collapse [21–23], are analysed using numerical simulations with the finite element approach.

It is widely agreed by researchers that IMCs have a crucial role to play in steel MBS' structural performance, and many claim the lack of robust and well-tested connections between modules is a significant limitation in their ability to resist gravitational loadings, horizontal actions, and to maintain stability despite abnormal events [24–28]. Discrete nature of connectivity in steel MBSs cause additional stresses and sometimes create complex stress paths, as complex connections between modules using bolts, plates, and shear keys.

Following localised structural damage caused by accidental loads such as explosions and impacts, structural robustness refers to the ability of a structure to resist progressive collapse [29]. Several standards and codes address robustness design, including Eurocode [29], DoD (U.S.) [30] and GSA (U.S.) [31]. When vertical load-bearing components of multi-story buildings are damaged by extreme events such as explosions and impacts, these buildings may face serious threats. In the event of a

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local failure, the load-transferring path may be interrupted, resulting in progressive collapse of the remaining structure [32]. However, it has been reported that modular buildings exhibit greater structural redundancy than conventional structures with continuous framing, which can enhance their ability to redistribute loads during extreme events, potentially improving their performance against progressive collapse [33–35]. This distinction underscores the need for comprehensive research into the robustness of steel MBSs to better understand their behaviours and structural performance under such loads. In principle, modules and inter-module connections (between modules) should provide collapse resistance in post element or module removal scenarios.

1.1. Background review

1.1.1. Robustness performance

Numerous cases of progressive collapse has been highlighted in the literature, while more attention has been directed to the causes and mitigation methods. The number of studies conducted on modular building systems, however, is limited. For instance, Thai et al. [36] presented an investigation on the progressive collapse and structural robustness of modular high-rise buildings under a sudden module removal scenario. As a result of the study, in modular high-rise buildings, lateral bracing systems and inter-module connections play a crucial role in ensuring the overall stability and robustness. Lawson et al. [37] explored the various failure scenarios and proposed a logical approach to structural integrity of modular buildings. The removal of a corner support revealed that the torsional stiffness of the box is important for distributing loads away from damaged sections. Luo et al. [38] evaluated the structural robustness and collapse mechanism of a medium-height steel modular building based on removal of a base corner module. According to the results, steel modular buildings could be made collapse-resistant by increasing in the number and capacity of supporting posts, use inter-module connections with higher rotational stiffness and capacity, and use more longitudinal bracing walls. Chua et al. [22] investigated the robustness of steel Prefabricated Prefinished Volumetric Construction (PPVC) high-rise buildings under column removal scenarios by conducting non-linear numerical analysis of a 40-storey steel frame PPVC building. It was found that due to the high redundancy of structural elements, PPVC high-rise buildings are robust enough to prevent progressive collapse. Alembagheri et al. [21] assessed gravity-induced progressive collapse of steel modular buildings focusing on the effects of inter-modular connections under module loss scenarios. In comparison to conventional buildings, the modular buildings were shown to be able to offer high levels of robustness and collapse resistance.

1.1.2. Impact of inter-module connection (IMC) on robustness performance

A simplified model of inter-module connection was proposed by Zong et al. [39] to capture the collapse properties of the modular steel constructions (MSC) by comparing it with a substructure push-down test. They found that when a side-modular unit failed, the inter-module connections of all four stories were damaged due to excessive overturning moments, causing the span to collapse. The dynamic response of corner-supported modular steel buildings with a core wall system under progressive collapse scenarios was examined by Haji Rezaie et al. [40]. As a result, they realised that softening the horizontal inter-module connections can worsen or improve the performance of the remaining modules in the event of progressive collapse caused by gravity, and the core walls improve the robustness of MSBs. Lacey et al. [41] provided a review of the modular building's structural response under various actions and assessed the vertical and horizontal connectivity capabilities of eighteen IMCs. Peng et al. [42] evaluated lateral performance of multi-story modular buildings with the effect of a tenon-connected inter-module connection being taken into account. In this study, a simplified joint model was created using Timoshenko beam elements and spring elements reducing computational costs and making system-level analyses practical. A study by Chan et al. [43] focused on different inter-module connections under a corner column removal scenario. There was evidence that the type of connection between modules affects the joint properties between beam-columns, and this has an impact on the progressive collapse resistance of modular structures. Experimental studies by Chen et al. [44] investigated the behaviour of an inter-module connection under lateral load, and argued how welding quality and stiffeners affected stiffness, strength, and ductility.

1.1.3. Impact of height on robustness performance

Despite research efforts to understand the response of structural systems to progressive collapse, there are very few studies that have examined the influence of building height on the robustness of modular building systems. From a risk assessment perspective, only a limited work has been conducted to correlate structural features at a global and local level with modular building robustness. For example, a recent study performed by Chua et al. [45] on collapse behaviour of lateral braced steel modular buildings. In this study, response behaviour and force transfer mechanism of low-to high-rise modular buildings were investigated. For modular steel buildings with a height ranging from 5 to 30 stories, the effect of removing columns was studied. Using nonlinear dynamic analysis, it was determined that the low- to high-rise modular buildings with lateral braces had sufficient resistance to progressive collapse when columns were removed. Under different sudden column loss scenarios, Peng et al. [46] studied the progressive collapse resistance of multi-story modular buildings utilising corner-supported composite modules. According to the dynamic analysis, 4-story building prototypes remained elastic while 12-storey counterparts develop partial yield at the joints; however, under design loads, the building prototypes remained stable.

While these advancements have been made, there remains a limited focus on comparative studies of different connection types and heights under different extreme loading conditions, which represents a major area for further investigation. Consequently, in the present study, the impact of the IMCs and building height on the potential of progressive collapse of the steel MBSs is investigated using nonlinear static analysis. The alternate load path (ALP) method, which is a threat-independent methodology, is employed for low- to high-rise modular buildings focusing on the effects of IMCs. Although many different IMCs have been proposed in the past several years, two different types of IMCs: 1) bolted connections and 2) post-tensioned rods are herein discussed, which have been proposed or adopted in [18,47]. To achieve these goals, 5-, 10-, and 15-story steel modular building are established using finite-element OpenSees software [48,49]. Column removal is conducted in this study as it is more likely to occur under accidental event instead of module removal. The robustness performance of the finite element (FE) model is assessed by considering instant loss of first story column at the middle location under nonlinear static analysis. Thereafter, the progressive collapse resistance of different types of IMCs and building height of the steel modular building systems are discussed. Then the failure criterion was precisely defined by considering the maximum dynamic increase factor (DIF) of the pushdown curve. Finally, the fragility of the structure can be easily determined using the probabilistic demand models and equivalent extreme value events. The fragility indices of both the intact and damaged structures allow for precise quantification of the structure's robustness. By highlighting the robustness of the MBSs, it is expected that findings in this study will contribute to the development of more resilient and safer steel modular buildings.

2. Fragility-based assessment framework

The objective of this approach is to determine the probability of an engineering demand parameter (EDP) exceeding various levels for a given design limit state at specific intensity measure (IM). This computation is essential for performance prediction, as it accounts for both intrinsic randomness - a measure of the inability to fully interpret structural elements, and uncertainty - a measure of errors that arise due to calculating models and procedures [50]. Using notional element removal to simulate the sudden loss of a structural component, the impact of the resulting changes in load distribution on EDPs is studied at various intensities. As a result, the probability of failure can be evaluated, clarifying how notional element removal and intensity measures relate to the robustness of the structure.

The particular method is employed to estimate the probability of failure, given that the pushdown method utilises the deterministic DIF of 2, in accordance with guidelines such as DoD (U.S.) [30] and GSA (U.S.) [31]. This approach aims to simulate and analyse the progressive collapse capacity of low- to high-rise steel modular building systems with varying IMCs.

Through incrementally applied vertical loads, the pushdown analysis method employed in this study approximated the structural behaviour under sudden column removal scenarios. While the notional element removal is inherently dynamic, the pushdown approach, coupled with a DIF of 2.0, provides a static representation that captures the key aspects of the dynamic response. By conducting such analyses, it is possible to determine the relationship between the degree of progressive collapse load factors (DIF) as IM and the response of the structure in terms of engineering demand parameter (EDP=VD). The results of the analyses are used to derive probabilistic demand models that relate uncertainties in gravity loads to structures' response variation.

Cornell et al. [51,52] demonstrated that conditional seismic demands can be modelled using a lognormal distribution, as shown in Eq. (1). In this study the parameters are transferred from seismic demands to progressive collapse scenario. However, the inherent concept of the close-formed approach remained intact.

$$P[VD \geq vd|\mathrm{DIF}] = 1 - \Phi\left(\frac{ln(vd) - ln(\eta_{VD|\mathrm{DIF}})}{\beta_{VD|\mathrm{DIF}}}\right) \tag{1}$$

where $\Phi(.)$ is the "standardised" Gaussian cumulative distribution function, $\eta_{VD|\mathrm{DIF}}$ is the median value of VD given DIF, and $\beta_{VD|LF}$ is the logarithmic standard deviation or dispersion of the VD conditioned on the DIF or standard deviation of the model error. $P[VD \geq vd|\mathrm{DIF}]$ at DIF levels for VD limit states generates fragility curves, indicating the usage of probabilistic demand models.

Eq. (1) can be translated further into lognormal space, as shown in Eq. (3), employing Eq. (2) that states the relationship between demand and DIF in the power form.

$$\eta_{VD|\mathrm{DIF}} = a(\mathrm{DIF})^b \tag{2}$$

$$ln(\eta_{VD|\mathrm{DIF}}) = b.ln(\mathrm{DIF}) + ln(a) \tag{3}$$

where a and b are constant parameters of the linear regression and ln(a) is the vertical intercept and b is the slope constant.

To generate data for regression, a set of 600 steps were used to incrementally increase the load factor and perform pushdown nonlinear static analyses on the selected MBS models. As the vertical displacement increased, there was a corresponding increase in load factor (DIF), indicating heteroscedasticity of response in terms of load factor.

The N demand quantities are plotted against the load factor to estimate the regression parameters and the dispersion term, as shown in Eq. (4):

$$\beta_{VD|\mathrm{DIF}} \cong \sqrt{\frac{\sum_{i=1}^{n}\left(ln(vd_i) - ln(\eta_{VD|\mathrm{DIF}})\right)^2}{n-2}} \tag{4}$$

where vd_i represents the i[th] realisation of VD from the nonlinear time history analysis, and n represents the number of analyses.

To provide more detail, Fig. 1 presents the framework of the fragility-based assessment of MBSs under a progressive collapse scenario. This framework can be utilised for developing probabilistic demand models for other scenarios.

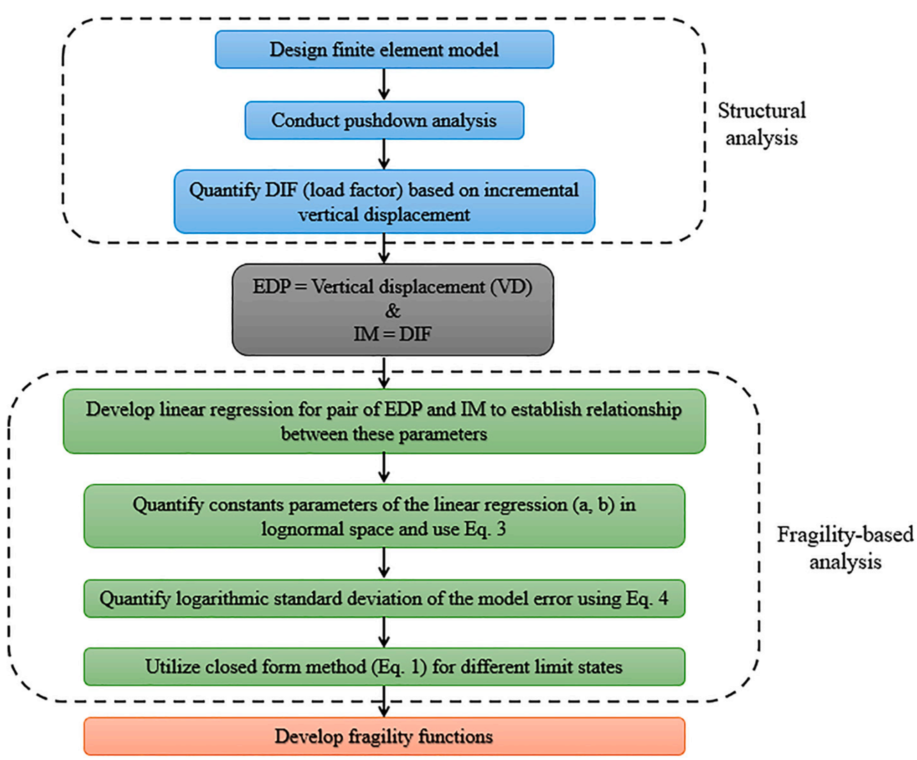


Fig. 1. Framework of fragility-based approach.

3. Case study and assumptions

3.1. Global structure

This study uses a steel modular building system consisted of corner-supported modules connected together at corner joints. In compliance with AISC [53], steel MBS braced frames with five bays in each direction were designed for low- to high-rise buildings. Fig. 2(a) shows a 3D view of the 10-story steel MBS studied. Fig. 2(b) illustrates the module dimensions of 6 m × 3.5 m × 3 m (length × width × height). Square hollow sections (HSSs), extensively used in MBS construction, are used for all columns, beams, and braces. Due to the offsets in column centrelines and the small spacing between consecutive columns, each module is modelled with horizontal axis gaps of 350 mm and vertical axis gaps of 200 mm. Each of the selected sections was identified by the same mechanical properties, such as a 200 GPa modulus of elasticity, 350 MPa yield strength, ultimate strength of 450 MPa, and a 0.3 Poisson's ratio.

For the design, Table 1 provides superimposed dead loads applicable to items such as floors, roofs, corridors, and ceilings, as well as live loads and snow loads.

To conduct the nonlinear analysis, a two-dimensional model of the MBSs is created using OpenSees software. A Menegotto-Pinto material with 1 % isotropic strain hardening (*Steel02*) was used to simulate the nonlinear material behaviour of elements. Beams and columns are represented using the *forceBeamColumn* element option, which has fibre sections and five integration points. In addition, the *dispBeamColumn* element is used to model brace elements, which have fibre sections and three integration points. Geometric nonlinearity is accounted for by the P-delta effect. An efficient macroscopic FE approach is deployed to represent the P-delta amplification effects induced by the gravity system in the 3D configuration using the leaning column concept. An axial load is applied to the leaning column equal to the sum of the gravity loads acting on the internal gravity columns in each story. Due to the absence of gravity columns in the two-dimensional model, their action is represented by a pin-ended rigid column linked rigidly to the frame. A discretised brace is constructed by using eight elements between work points, each with three integration points and an initial out-of-plan imperfection of L/500. A single and multiple springs aligned with the brace and located at the ends of the gusset plates were considered in order to capture the out-of-plane rotational behaviour of the gusset plates at the connections. Floors are constructed with primary beams, secondary beams (joists), metal decks, and concrete toppings. For the floor system, a composite concrete steel deck system with a weight per unit volume of 23.56 kN/m^3, compressive strength of 35 MPa, and slab thickness of 120 mm is used. As for the ceiling, Autoclaved Aerated Concrete (AAC) flooring planks are utilised, making use of the light-weight concrete which has a density of 6.52 kN/m^3, a compressive strength of 4 MPa, and a slab thickness of 200 mm.

In the first step of the analysis, the Krylov-Newton solution algorithm is used. Changing the solution algorithm is necessary if convergence cannot be achieved. Several algorithms were used in this study to solve the adaptive solution strategy, including Krylov-Newton, Raphson-Newton, ModifiedNewton, and Newton with Line Search.

Table 1
Superimposed dead, live and snow load applied for the design of MBS.

Structural components	Load type	Load
Floor slab	Superimposed dead load	0.75 KN/m^2
	Live load	2 KN/m^2
Ceiling slab	Superimposed dead load	0.7 KN/m^2
	Superimposed dead load	0.32 KN/m^2
Roof	Live load	1 KN/m^2
	Snow load	1 KN/m^2
Corridor	Live load	4.8 KN/m^2
External Floor beam	Dead load	1.5 KN/m

3.2. Inter-module connections assumptions

Under extreme loading conditions, such as progressive collapse, the performance of joints between modules is critical, as these connections directly influence the structural integrity and load distribution. The beams and columns of a module are designed to be rigidly connected to ensure robustness and stability. This simple joining technique is used to tie adjacent modules (e.g., horizontal tying) as well as upper and lower

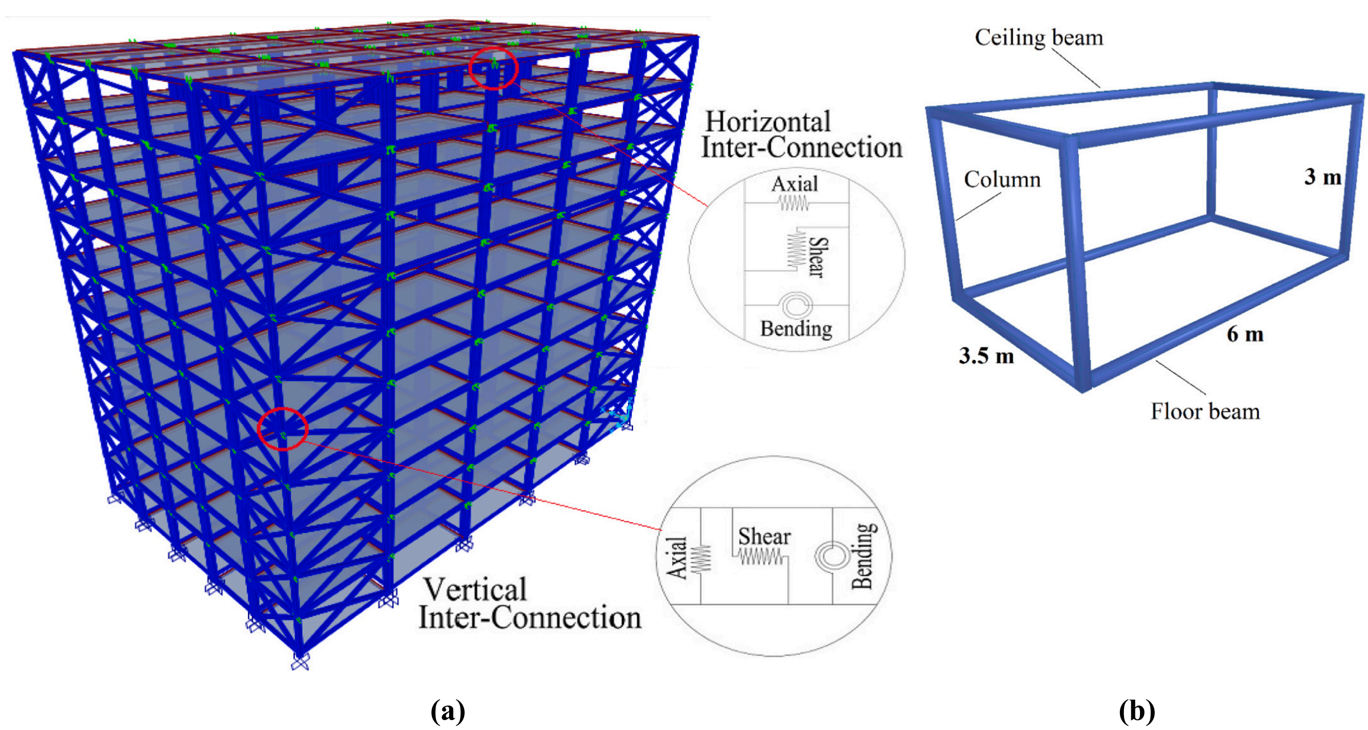


Fig. 2. 3D view of (a) 10-storey steel modular building system (MBS) and (b) Single module.

modules (e.g., vertical tying), as shown in Fig. 3. In addition, the behaviour of connections can be represented appropriately in global simulations using the translational spring model.

Throughout the past decade [24], researchers have proposed numerous types of IMCs. In this study the impacts of two types of connections including bolted IMCs and post-tensioned rod IMCs for steel modular building systems are discussed. The adopted IMCs are applied to the steel MBSs shown in Section 3.3. It is expected that the embraced IMCs can truly represent the common features of this class of connections.

3.2.1. Bolted IMCs

A sample of bolted vertical and horizontal IMCs is described by Styles et al. [47], as shown in Fig. 4(a). Horizontal and vertical IMCs were modelled to link modules at their corners. Vertical interconnections were characterised by force-displacement data. The axial, shear, and moment behaviours of the horizontal and vertical IMCs were determined and applied to the MBSs. To represent their structural behaviour accurately, connections are considered to fail when their force or displacement capacities are exceeded. By taking this step, the simulation accounts for the reduced capacity of the system after a failed connection, ensuring that the model accurately represents the structural behaviour after failure. Therefore, the nonlinear behaviours of the IMCs were considered, as indicated in Fig. 5(a-e). Load-displacement and moment-rotation curves were derived from the FE models. To ensure that the derived curves accurately reflect realistic connection behaviour, the finite element analysis (FEA) results were compared with experimental results from literature.

The vertical IMC illustrated in Fig. 4(a) shows a typical column splice connection, in which there is an end-plate welded to the column end, similarly to what is shown in Table H.38 in the JSC [54]. As a means of connecting adjacent columns, bolts and nuts are used to create horizontal tie connections (see Fig. 4(a)). To attach each side plate to the column face, bolt holes are pre-drilled. For the purpose of determining the geometry of the connection, Chapter 8 of AS4100 [55] was used. As part of the modelling process, grade 350 steel and bolts of class 10.9 M24 were taken into account [47].

3.2.2. Post-tensioned rod IMCs

Chua et al. [18] studied a sample of post-tensioned rod as shown in Fig. 4(b), consisting of horizontal and vertical inter-module connections. Vertical continuity can only be achieved by using the vertical rod since there is no continuous column between upper and lower modules. Hence, the columns are designed to withstand compressive loads, while the vertical rods are designed to withstand tension and prevent the modules from separating vertically under high lateral forces. With the help of the shear key, located on the top plate on the upper column, bearing action is applied by the upper column to resist the shear force between the upper and lower modules. A horizontal tie plate bearing against a shear key transfers horizontal force from one module to the next.

The stiffness parameters of the vertical and horizontal rod connections with shear-key are elaborated in Table 2. It it worth noted that the horizontal and vertical IMCs acted as pins to prevent the transfer of moment between columns and tie plates.

As per the calculations conducted in Eq. (5), the axial compressive and tensile stiffness of the horizontal and vertical mechanism have been determined. It is important to note that the rotational stiffness has been found to be negligible due to the axial elongation of the tension rod [18].

$$K_{axial} = \frac{AE}{L} \tag{5}$$

where, A equals to cross sectional area, E equals to Young's Modulus of member, and L is considered as axial length. Also, the shear stiffness of the inter-module connections is calculated as shown in Eq. (6).

$$K_{shear} = \frac{AG}{L} \tag{6}$$

where, G equals to Shear Modulus of material and L is considered as axial length.

3.3. Model's height assumptions

The impact of building height on the progressive collapse resistance of steel MBSs under specified column loss events was studied in three buildings with 5, 10, and 15 stories. Building models were first designed

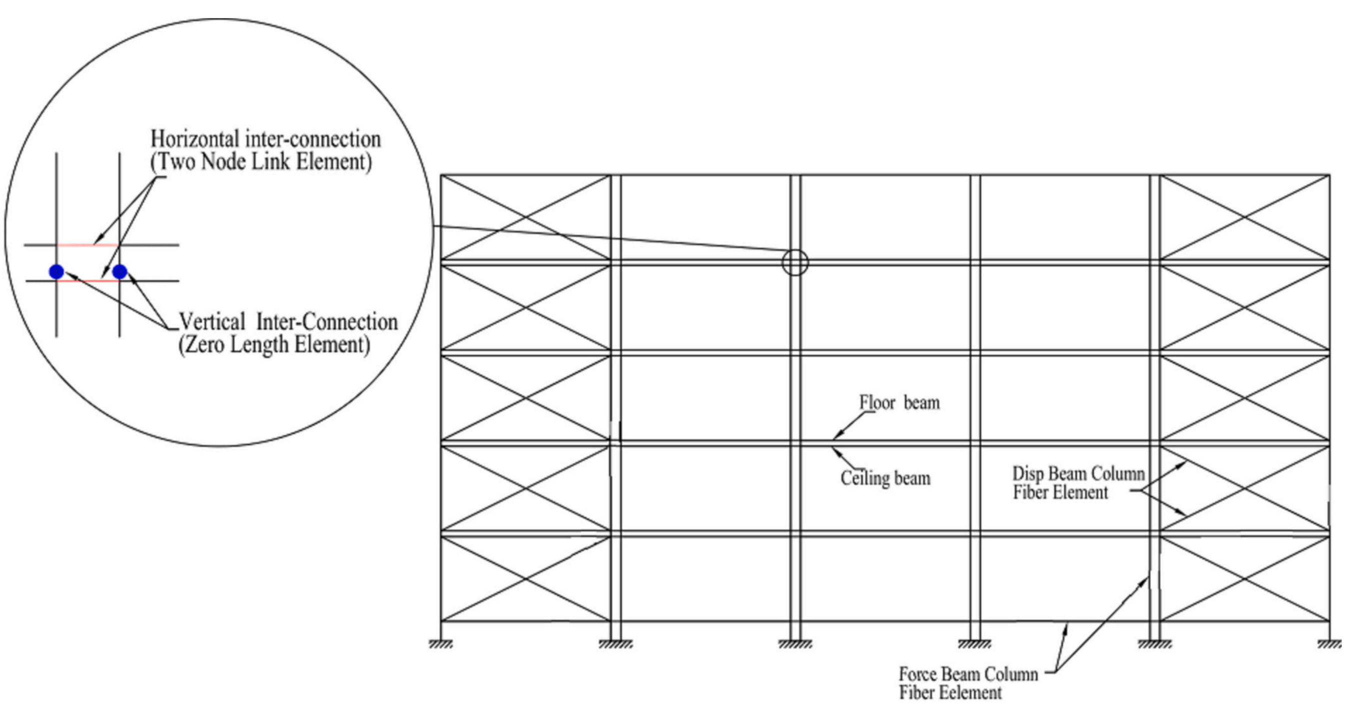


Fig. 3. Visual model of horizontal and vertical connections used in this study.

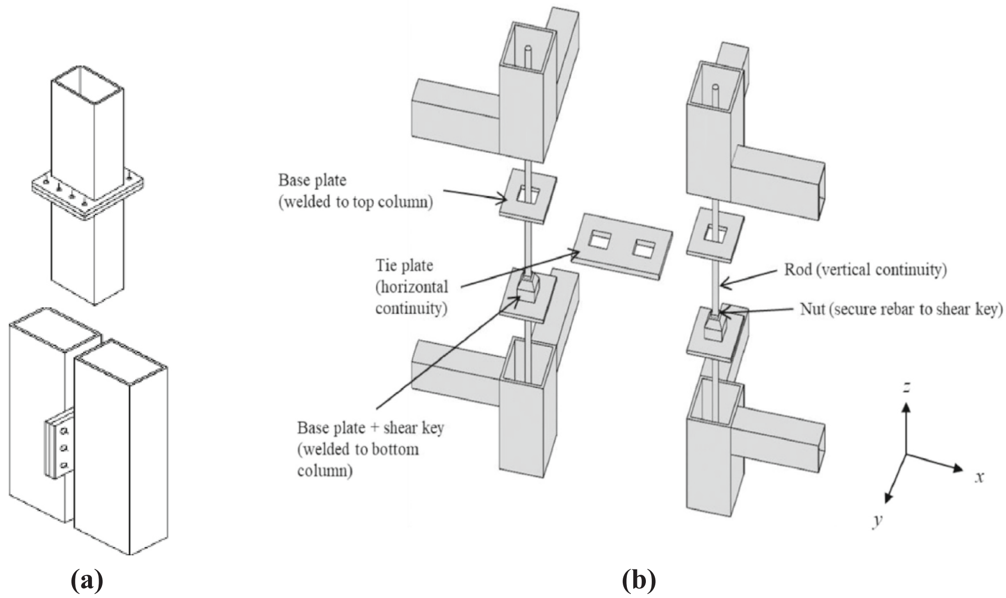


Fig. 4. Two types of IMCs used in this study: (a) Bolted IMCs [47], and (b) Post-tensioned rod [18].

based on AISC [53] and optimised in commercial SAP2000 software; thereafter, the two-dimensional models, representing a true behaviour of the 3D models, were developed in OpenSees software. Fig. 6 illustrates the building models used in this study.

Detailed information regarding the member sizes used in the modular building models can be found in Table 3.

3.4. Validation

To validate the proposed method, a 2D static test was performed using experimental data from the National Institute of Standards and Technology (NIST) [56]. The setup, as shown in Fig. 7.a, involved beam-column assemblies consisting of three columns and two beams connected through welded unreinforced flange-bolted web (WUF-B) connections. The numerical simulation was carried out using OpenSees, an open-source finite element software, with a centreline-to-centreline modelling approach. To model the beams and columns, distributed plasticity fibre-based nonlinear beam-column elements were employed. These elements, characterised by sectional fibre discretisation, are adept at capturing nonlinearity along their length. For the connection modelling, the Krawinkler model was used [57,58], which has been widely adopted for simulating the behaviour of moment connections under seismic loads [53]. The Krawinkler approach effectively represents both the panel zone deformation and the rotational flexibility of the connection.

The steel material properties were calibrated to match the experimental data and modelled using the *Steel02* material in OpenSees. Sectional details, dimensions, and applied loads (comprising dead and live loads) were meticulously replicated from the experimental setup to ensure accuracy [56]. In the numerical model, the central column was removed (Fig. 7). A vertical load of 2,669 kN, simulating the hydraulic ram from the experiment, was applied at the top of the central column. Additionally, an incremental displacement of 508 mm was applied at the mid-span. Static and displacement-controlled analyses were used to evaluate the structural response under these conditions.

Fig. 7.b presents the relationship between vertical force and displacement at the centre of the span where the column was removed. In the initial elastic phase, there is a strong agreement between the experimental and numerical results, with a stiffness of approximately 10 MN/m. In the experiment, yielding occurred at 500 kN with a displacement of 0.05 m. However, in the numerical simulation, yielding was observed at a higher load of 600 kN and a displacement of 0.07 m. Both curves display a hardening behaviour in the post-yield phase. The differences in post-yield behaviour between the experimental and numerical results can be attributed to several factors. Numerical models often simplify material properties, connection details, and boundary conditions, which may not capture the full complexity of real-world behaviour. Additionally, strain hardening and geometric non-linearities, such as local buckling or imperfections, can vary between the experiment and simulation. These factors contribute to the observed variations in the post-yield phase. Moreover, Fig. 7.c illustrates the cumulative energy at the centre column, showing similar overall trends between experimental and numerical results, with minor variations in specific areas. The cumulative energy, derived from the area under the force–displacement curves, represents the work done by the hydraulic ram during displacement, reflecting both the energy stored in the elastic phase and the energy dissipated in the plastic phase. Despite differences in the force–displacement curves, the energy curves are nearly identical, indicating that while the numerical model effectively captures the early elastic behaviour, it underestimates the non-linear effects and energy dissipation in the post-yield phase.

4. Numerical analysis methodology

Multi-story modular buildings are frequently studied using nonlinear static analysis (pushdown) due to the complexity and expense of conducting experimental tests. This study evaluates the robustness of steel MBSs under a column loss scenario using this approach. This section discusses the methodology and analysis procedures used in the present study to conduct a comprehensive analysis.

4.1. Methodology and analysis procedures

Nonlinear static analysis (pushdown) was employed to assess the progressive collapse resistance of the steel modular buildings in an accidental event employing ALP method. Due to the complex nature of nonlinear dynamic analysis, it proves to be an inefficient method to employ for practical applications, and as a result several procedures as well as explicit expressions have been proposed in the literature to approximate the dynamic component with a simplified static analysis [59–63].

The nonlinear static pushdown analysis, conventionally, was

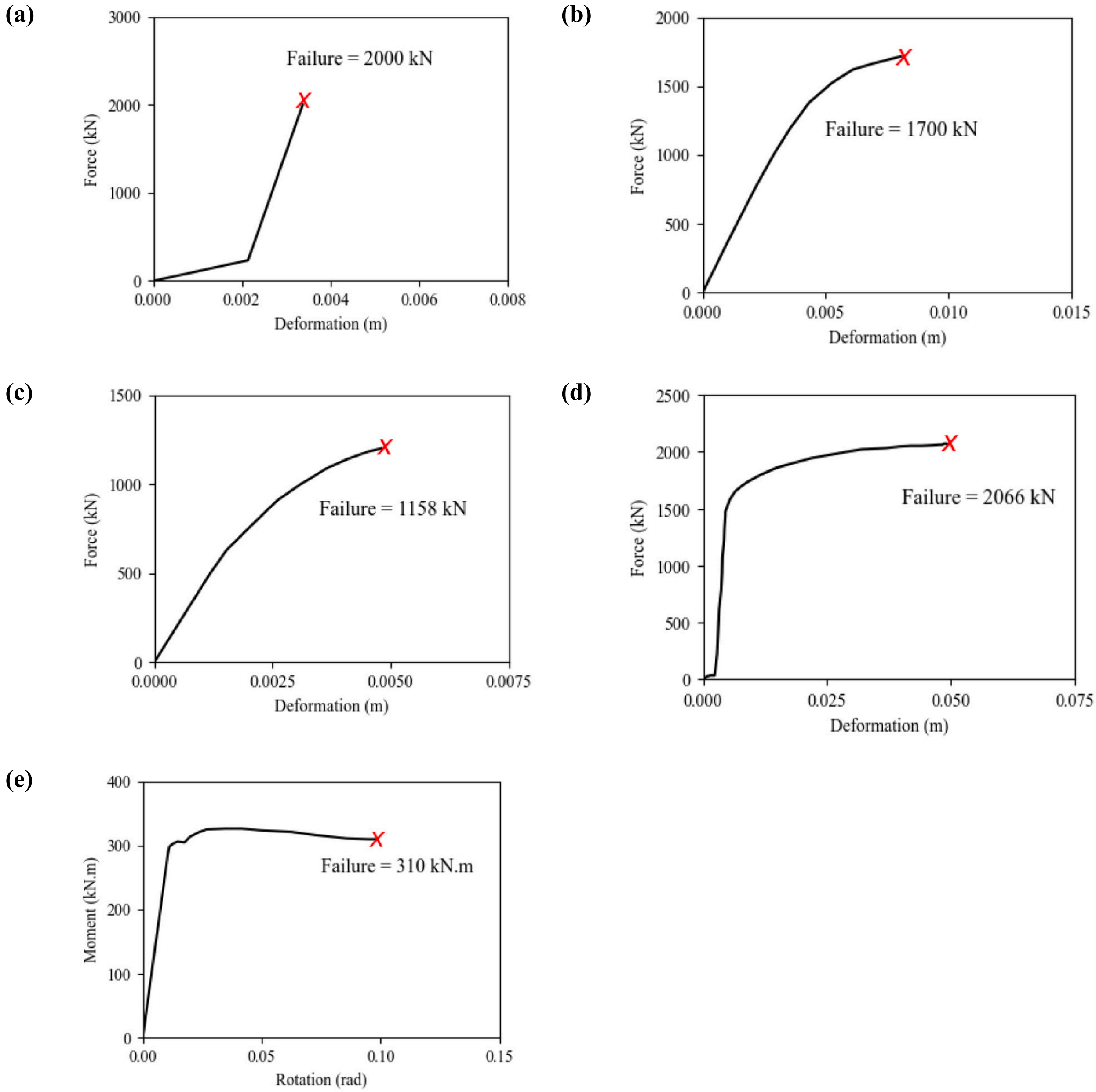


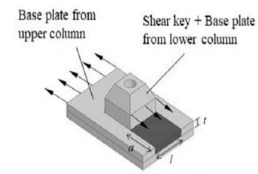
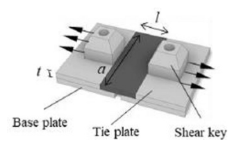
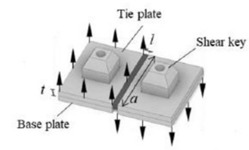
Fig. 5. The nonlinear behaviour of (a) Vertical connection in shear direction, (b) Vertical connection in axial direction, (c) Horizontal connection in axial direction, (d) Horizontal connection in shear direction, (e) Horizontal connection and Vertical connection in moment direction [47].

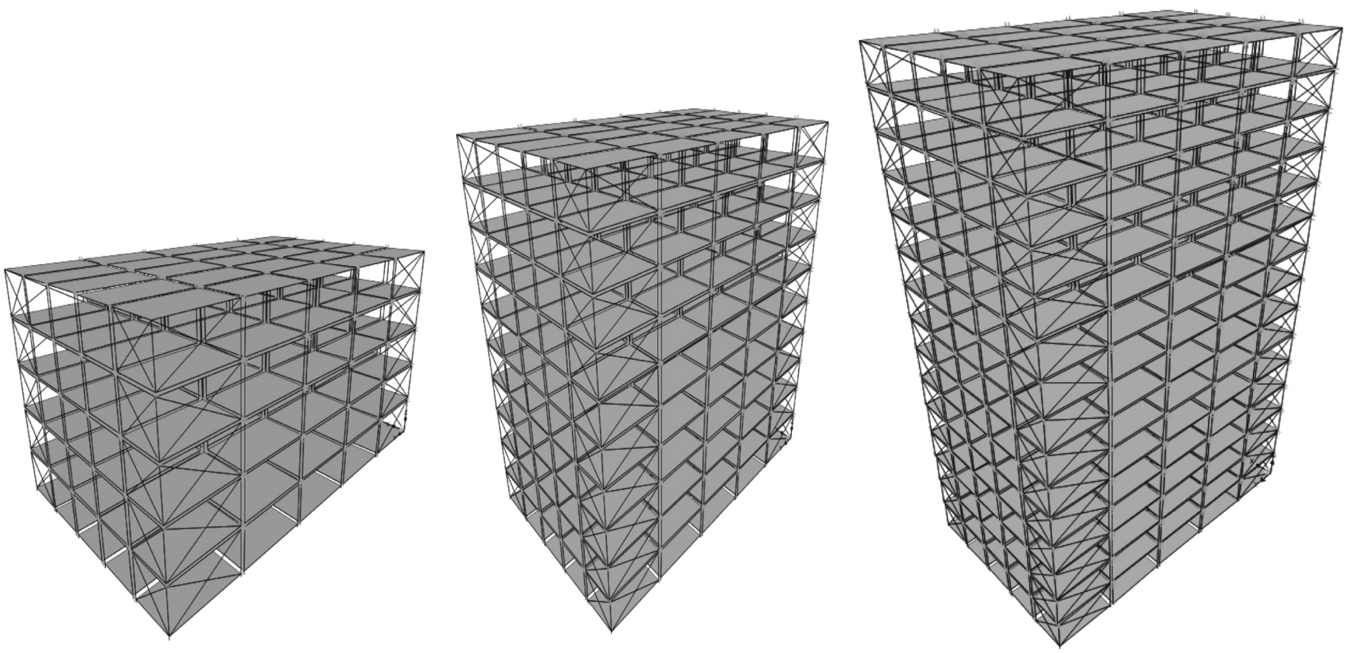
replaced by a static analysis with a constant DIF of 2.0 for gravity load, above the damaged components [64–66]. According to several guidelines, such as DoD (US) [30] and GSA (US) [31], a formula has been implemented that is based on the type of structure and the rotation limit that can be applied to it. Pushdown analysis is performed in different methods: uniform pushdown; bay pushdown; and incremental dynamic pushdown. Bay pushdown method is adopted in this study. In this method the intended column at ground level is initially removed as shown in Fig. 8. During the loading procedure, progressively increasing vertical loads are applied to the damaged bays until it reaches maximum load, or until the structure collapses. For nonlinear static analysis, GSA [31] provides guidelines on how to intensify the maximum load in nonlinear static analysis as shown in Eq. (7):

$$\text{Intensified loading} = \text{DIF} \times [1.2\,\text{DL} + 0.5\,\text{LL}] \tag{7}$$

where DL refers to the dead load on the vertical axis, and LL to the live load on the vertical axis. DIF varies in each step of analysis for the bays supported by the removed column and has a value of 1.0 for the other bays, as shown in Fig. 8. Thereafter, the vertical displacement of the point above the removed column is recorded. The pushdown curve corresponds to the plot of load factor DIF against its corresponding vertical displacement. In DoD [30], 0.2 L is specified as a criterion for tie force, where L indicates a module span length. It is assumed that at relatively large deformations, a tensile catenary action will occur, which may result in beam or intra-module connection failures. It must be noted that no tensile catenary mechanism is observed in this study. However, it is imperative to consider the possibility of welding fracture and determine whether the weld can withstand extreme deformation, as observed in certain experimental tests [35]. Therefore, it is assumed that the displacement-control analysis is stopped when the vertical displacement of the node above the removed column reaches 0.6 m ($= 0.1 \times 6$m). It has been determined that the structure's maximum displacement capacity is 0.1 L. This value is deemed adequate for the formation of fully

Table 2
Stiffness parameters of vertical and horizontal post-tensioned rod IMCs [18].

	Axial stiffness (KN/m)			Shear stiffness (KN/m)	
	Compression	Tension			
Vertical connection	12.9×10^3	0.04×10^3		1.5×10^3	
Horizontal connection	11×10^3			24.3×10^3	



(a) (b) (c)

Fig. 6. 3D view of MBS case studies: (a) 5-story MBS, (b) 10-story MBS, (c) 15-story MBS.

Table 3
Section properties of elements.

	5-story MBS	10-story MBS	15-story MBS	
Member	Story 1–5 (mm)	Story 1–10 (mm)	Story 1–10 (mm)	Story 11–15 (mm)
Column	HSS 127×127×8.9	HSS 203×203×11.8	HSS 229×229×14.8	HSS 178×178×11.8
Brace	HSS 127×127×5.9	HSS 152×152×5.9	HSS 152×152×11.8	HSS 127×127×11.8
Floor beam	HSS 102×102×5.9	HSS 127×127×5.9	HSS 127×127×5.9	HSS 127×127×5.9
Ceiling beam	HSS 76×76×5.9	HSS 102×102×5.9	HSS 102×102×5.9	HSS 102×102×5.9

plastic hinges at the joints or members.

As shown in Fig. 8, pushdown analysis was performed for the removal of a single external column. This scenario was selected in order to evaluate the robustness of the modular building system. While the focus of this analysis is on the removal of external columns, future research will include additional scenarios, such as the removal of corner columns and internal columns, to investigate the behaviour of the structure under different failure scenarios.

5. Results and discussion

Comprehensive findings from the simulations of nonlinear static progressive collapse are presented in this section. This study focuses on the impact of a column loss scenario on low- to high-rise MBSs with different IMCs. When MBSs were subjected to the column loss scenario, a significant redistribution of forces was observed, which could have potentially jeopardise the stability of the MBS. However, the analysis revealed that the inter-module connections that remained intact were able to handle the new redistributed forces without exceeding their capacity. This is a crucial finding, as it indicates the robustness of the

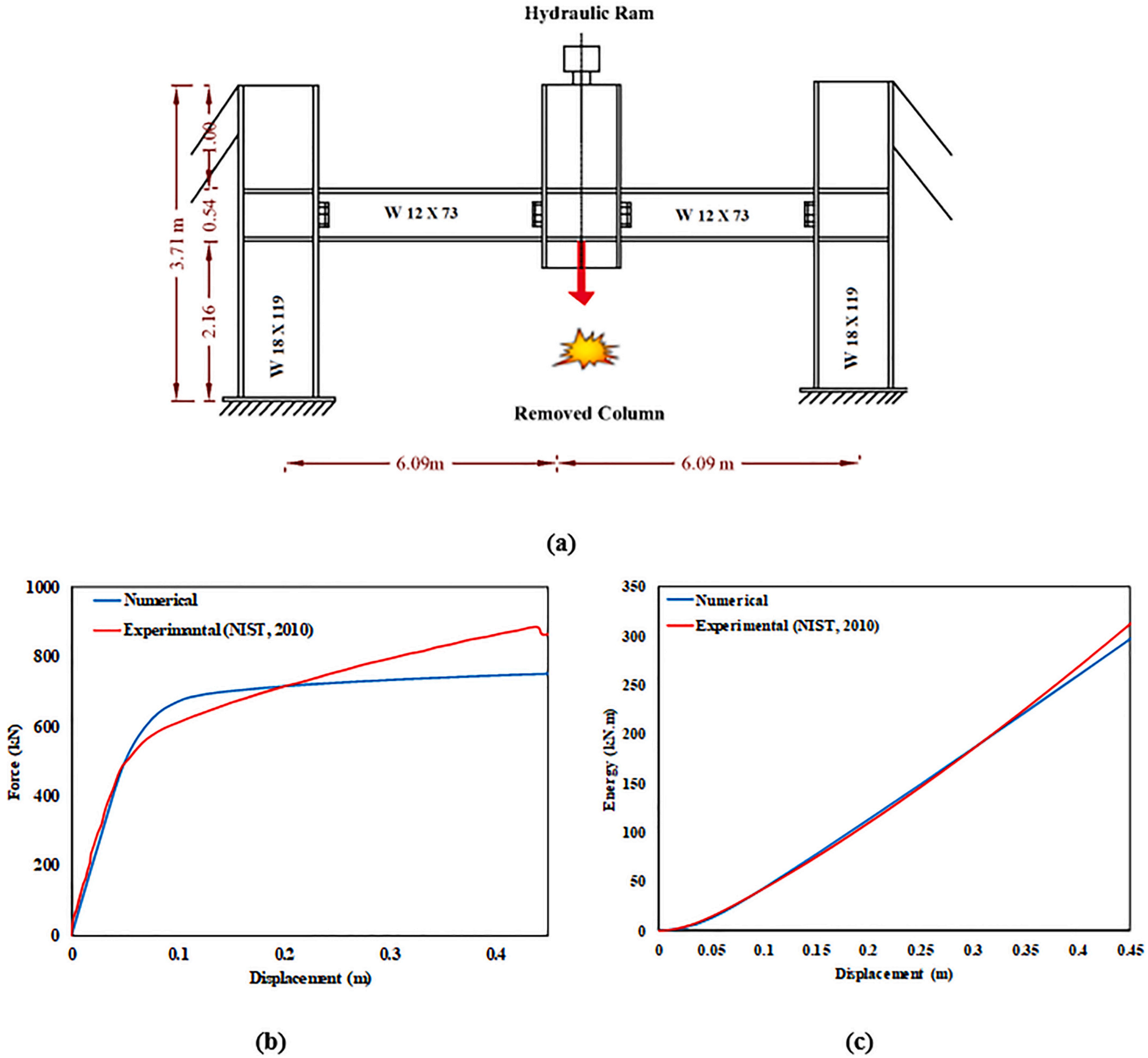


Fig. 7. Comparison of the experimental results and numerical results for progressive collapse scenario: (a) Details of test specimen, (b) Applied vertical load versus vertical displacements at center column, and (c) Cumulative energy versus vertical displacement at center column.

MBSs. In particular, the lateral inter-story drift ratios were investigated and the redistributed forces in the connections remained well within their capacity. This suggests that the MBSs can withstand the impact of column loss scenarios without suffering significant damage. The robustness assessments were also conducted, which provide valuable insights into the progressive collapse resisting performance of the low- to high-rise modular building systems with different IMCs under column loss scenario. Finally, the probabilistic demand models were developed for the studied MBSs with different types of IMCs. This can help utilise a range of DIF estimation instead of just a deterministic DIF for progressive collapse performance of MBSs.

5.1. Lateral inter-story drift ratios (IDR)

In vertical pushdown analyses, lateral drift ratios are important because they indicate the horizontal movements that occur following the removal of a column. In addition to vertical displacement, this lateral displacement provides a better understanding of the structure's response to progressive collapse and helps assess the overall stability of the structure. The lateral inter-story drift ratios of the 5-, 10-, and 15-story MBSs with different IMCs are shown in Fig. 9. The results indicate that the maximum lateral IDRs are increasing from low- to high-rise MBSs for both types of IMCs. In addition to this, it is worth noting that bolted IMCs tend to experience the highest IDRs at the mid-height of the building. On the other hand, post-tensioned IMCs generally experience their maximum IDRs at the top of MBS. It is important to mention that bolted connections allow for more lateral movement, leading to higher IDRs in low- and high-rise MBSs. This increased flexibility results in a "softer" lateral behaviour, especially at the mid-height of the building where cumulative deformations are greatest. On the other hand, post-tensioned connections in mid-rise MBSs experience higher IDR, resulting in softer lateral behaviour. In terms of the highest experienced IDR,

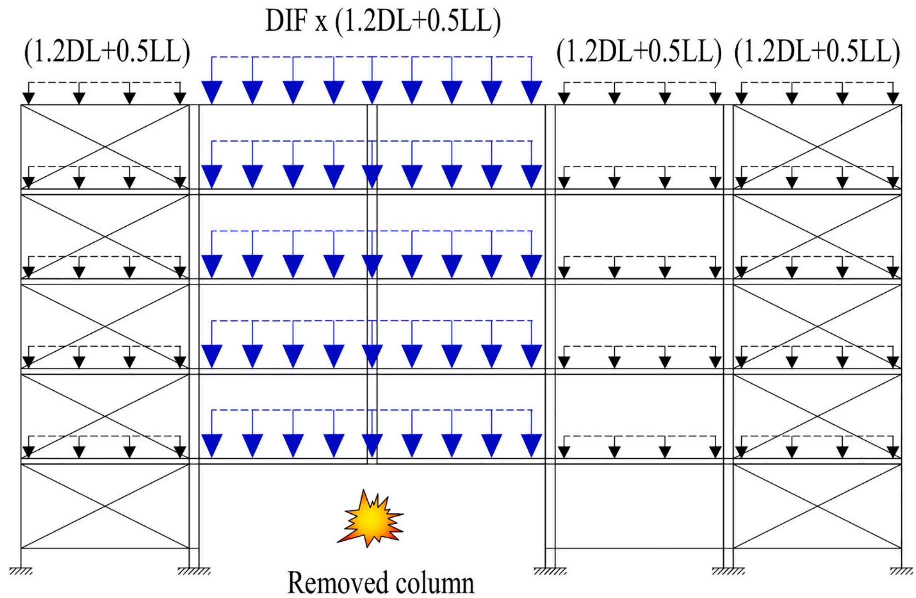


Fig. 8. Pushdown analysis of 5-story MBS model for progressive collapse scenario.

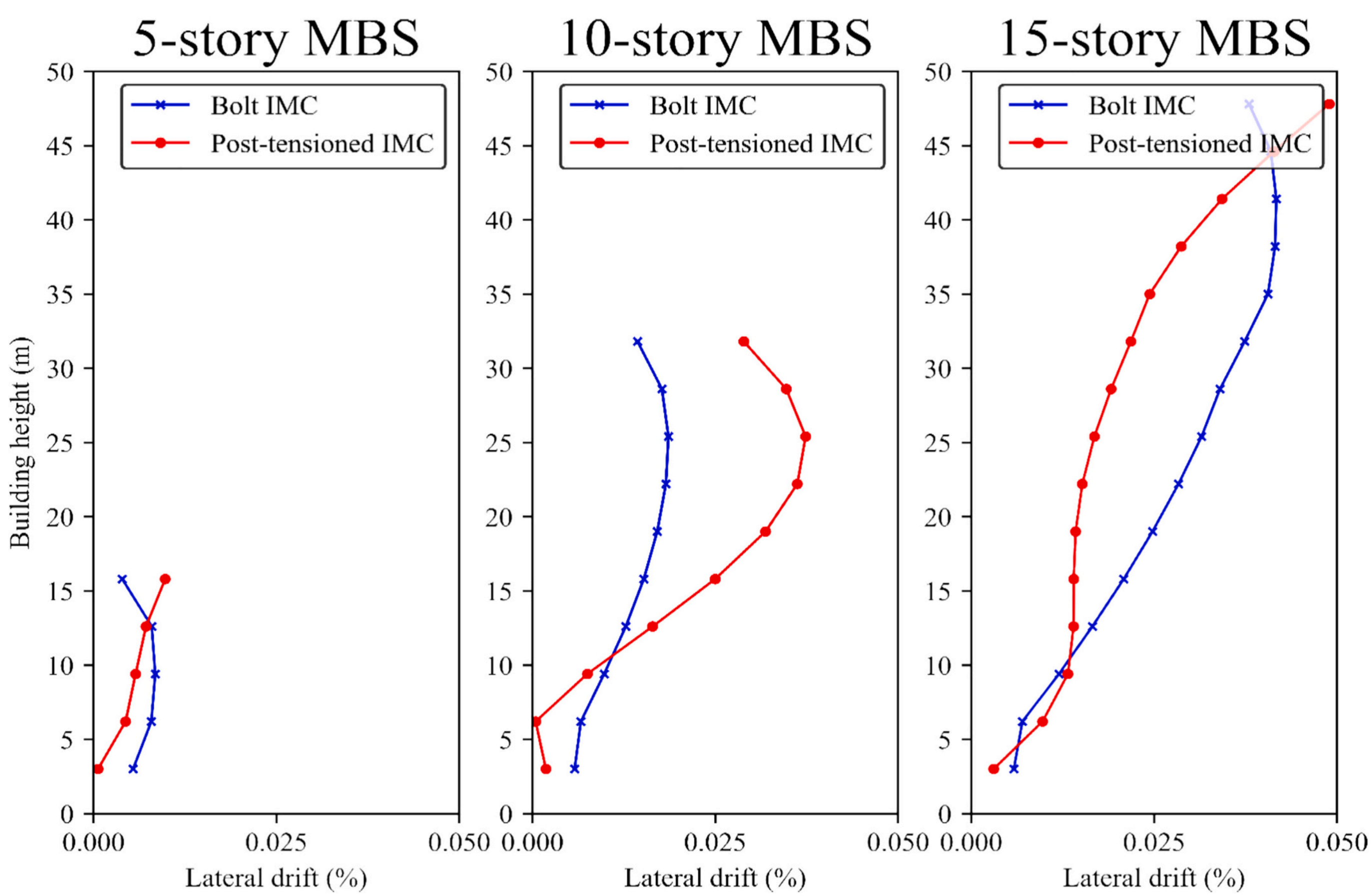


Fig. 9. Lateral inter-story drift ratios of the MBSs.

post-tensioned connections generally experience the highest IDRs, resulting in softer behaviour of low- to high-rise MBSs during load redistribution in a progressive collapse scenario. This difference in IMC type can have a significant impact on the overall strength and stability of the building, making it a crucial factor to consider during the construction and design process.

Regarding the height of the structures, bolted IMCs in the 15-story MBS exhibit noticeably higher lateral drift, particularly towards the top stories. As a result, taller structures may be at greater risk of failure modes such as story sway or localised instability due to inter-story movements. In contrast, the 5-story MBS exhibits a relatively small difference in lateral drift between the two connection types, suggesting that IMC type becomes more significant as height is increased, which is aligned with the previous study [36].

5.2. Inter-module connection (IMC) forces

5.2.1. Bolted IMC

Based on the results obtained from the nonlinear static pushdown analysis, Fig. 10 indicates the peak axial and shear forces that are present in the vertical bolted IMCs for each level of low- to high-rise MBSs. As a result of the analysis, the MBS's structural integrity and safety under load redistribution are revealed. However, despite significant changes in the distribution of forces within the structure following the removal of a column, the horizontal and vertical bolted connections remained intact and functional, without any signs of failure. This suggests that all building models exhibit a commendable degree of stability and possess sufficient resilience to endure scenarios involving the loss of individual column. It is observed that the lowest story in 5-, 10-, and 15-story MBSs experiences the highest value of axial forces. After analysing the available data, it was found that the vertical bolted links in MBSs of different heights experience varying peak axial forces. Specifically, MBSs with 5, 10, and 15 stories showed the highest axial forces of 107 kN, 335 kN, and 510 kN, respectively. It is essential to note that these values correspond to the models' lowest heights, which indicates a significant redistribution of force in the building connections. Nonetheless, the demand-to-capacity ratio (DCR) is comparatively low, measuring 0.06, 0.2, and 0.3 for the 5-, 10-, and 15-story MBSs,

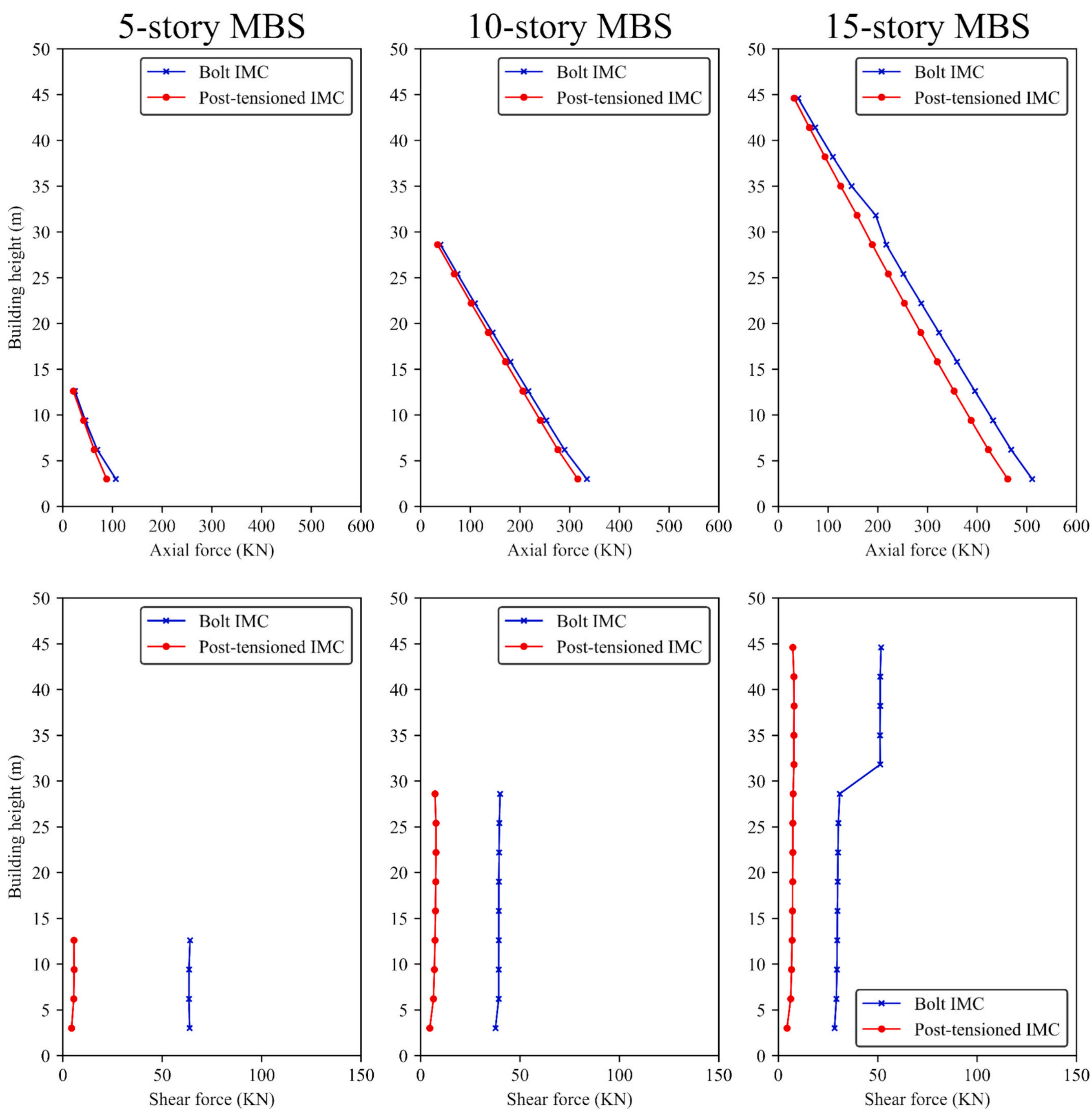


Fig. 10. Peak axial and shear forces in vertical IMCs for 5, 10, and 15-story MBSs.

respectively, with a total axial capacity of 1700 kN for the connections of these MBSs (as detailed in Section 3.2.1). Another key factor to consider is the shear forces in the vertical bolted links. In most MBSs, these forces tend to remain relatively consistent throughout the height of the building. However, when it comes to the 15-story MBS, this is not necessarily the case. This, in turn, means that the MBS's vertical links are capable of withstanding shear reactions during a progressive collapse scenario. When looking at MBSs with different heights, it is interesting to note how the shear forces vary. For instance, in MBSs with 5, 10, and 15 stories, the highest shear forces experienced were 64 kN, 40 kN, and 52 kN, respectively. The DCR for the 5-, 10-, and 15-story MBSs was 0.03, 0.02, and 0.026, respectively. Given the connection's shear capacity of 2000 kN, this indicates a substantial safety margin for the studied low- to high-rise MBSs.

As depicted in Fig. 11, the horizontal bolted IMCs experience the highest axial and shear forces across various levels of MBSs, ranging from low to high-rise buildings. It is noteworthy that the bottom floor typically endures the maximum amount of axial force, which is consistent with the vertical connections. It has been observed that the maximum axial force varies according to the number of stories. Specifically, in MBSs with 5-, 10-, and 15-story, the maximum axial force is 69 kN, 41 kN, and 39 kN, respectively. This result implies that horizontal bolted IMCs in low-story MBSs experience higher axial force compared to high-rise buildings. The DCR for the 5-, 10-, and 15-story MBSs was determined to be 0.06, 0.035, and 0.034, respectively. These results underscore the necessity of considering the number of stories in MBS design, as it influences the axial forces on horizontal bolted IMCs. Regarding the peak shear forces in horizontal bolted connections from Fig. 11, a zigzag pattern is observed, especially for the high-rise MBSs. This observation implies that the horizontal links do not possess an ample amount of shear capacity to sustain the cantilever force generated by the modules located above the eliminated one(s). This could

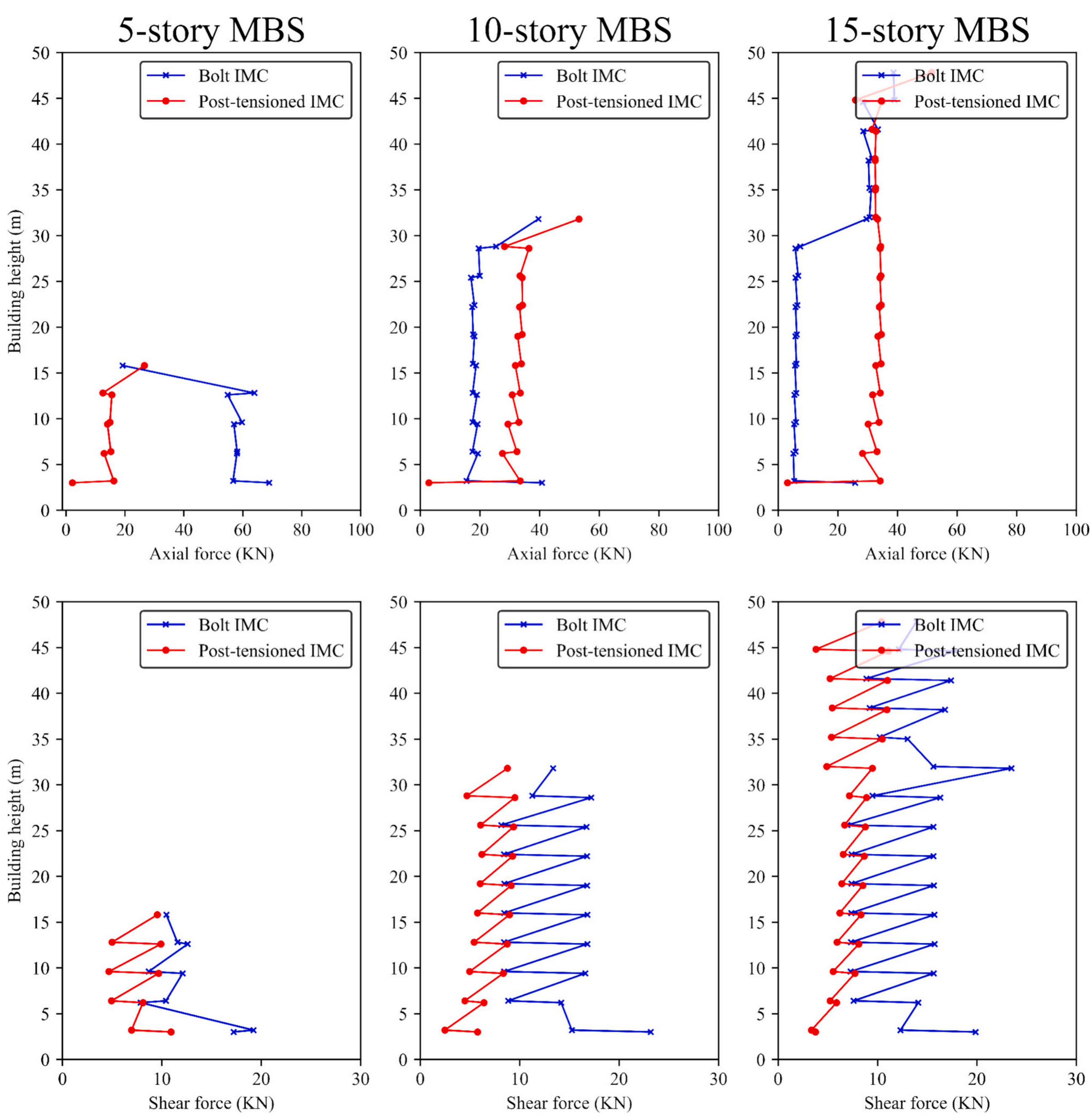


Fig. 11. Peak axial and shear forces in horizontal IMCs for 5, 10, and 15-story MBSs.

potentially lead to structural instability and pose a significant risk to the safety and structural integrity of the entire structure [67]. The maximum shear forces for the 5-, 10-, and 15-story MBSs were recorded at 19 kN, 23 kN, and 24 kN, respectively. Analysis revealed DCR values of 0.01, 0.011, and 0.012 for the 5-, 10-, and 15-story MBSs, respectively, indicating high safety margins for the connections and a low risk of failure.

5.2.2. Post-tensioned IMC

The nonlinear static pushdown analysis conducted on the MBSs provided valuable information about the structural behaviour of the buildings. The maximum axial and shear forces observed in both vertical and horizontal post-tensioned IMCs at different levels of low- to high-rise MBSs are presented in Fig. 10 and Fig. 11, respectively. The information presented in Fig. 10 suggests that the vertical IMCs in both types exhibit similar behaviour under axial loads and can tolerate comparable amounts of force. In addition, Fig. 10 provides a clear illustration that vertical bolted IMCs are subjected to significantly higher shear forces than post-tensioned IMCs. This is primarily attributed to the presence of vertical rods that are specifically designed to withstand axial loads and prevent the modules from separating vertically. The vertical rods, together with the shear key located on the top plate of the upper column, provide exceptional resistance against shear forces. The shear key, which is a unique component of post-tensioned IMCs, helps the upper column apply a bearing action, which prevents the upper and lower modules from separating. It is worth noting that the bearing action of the upper column is crucial in resisting the shear force between the upper and lower modules, and this is why post-tensioned IMCs offer superior shear behaviour.

Fig. 11 presents a comparison of the axial behaviours of horizontal bolted and post-tensioned IMCs. The results indicate that these two types of IMCs exhibit different behaviours under axial loads. Moreover, the behaviour of low-rise MBSs is significantly different from that of mid- to high-rise MBSs. In mid- and high-rise MBSs, the horizontal post-tensioned IMCs generally are subjected to higher axial forces than the bolted IMCs. However, in low-rise MBSs, the axial forces imposed on the horizontal bolted IMCs are greater than those imposed on the post-tensioned IMCs. After conducting a comprehensive analysis of the shear behaviour of horizontal bolted and post-tensioned IMCs, it has been determined that the bolted IMCs are subjected to significantly higher shear loads compared to post-tensioned IMCs.

The findings indicate that the axial and shear behaviour of vertical and horizontal connections in MBSs under progressive collapse is influenced by multiple factors, such as the height of the building and the type of IMC used. Specifically, bolted IMCs are seen to experience higher axial and shear loads during column loss than post-tensioned IMCs. Therefore, when designing MBSs, careful consideration should be given to both post-tensioned and bolted IMCs, considering the aforementioned factors. This will help ensure the safety and structural integrity of the building during extreme events.

5.3. Robustness assessment

It has been determined that all MBS models maintain sufficient robustness when subjected to the loss of a column in the considered scenario. As a result, it is possible to determine their capacity using static pushdown analysis. This method allows for a comprehensive calculation of the building's ability to withstand the effects of the column loss scenario, providing a more detailed analysis of the structure's robustness performance under these conditions.

Figs. 12 and 13 present a detailed analysis of the pushdown curves, specifically for different IMCs and low- to high-rise MBSs. The pushdown curves depict the incremental DIFs and the vertical displacements at the top point of the damaged bays. The data from the pushdown curves can be used to determine the robustness and capacity of the structure, providing insight into its overall stability. In the pushdown curves, three different stages can be distinguished. Each stage represents a different

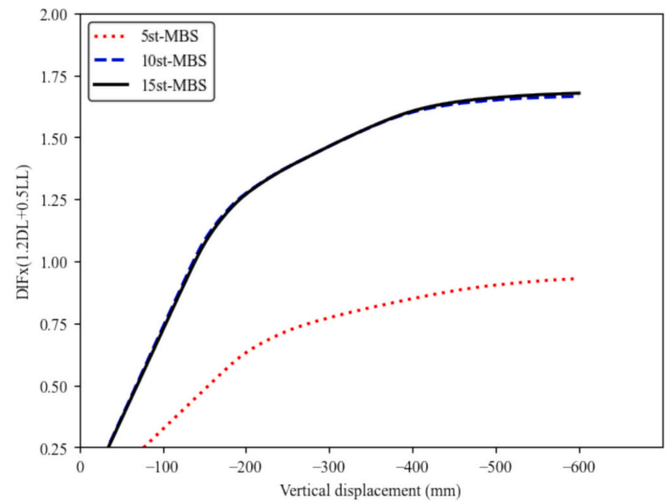


Fig. 12. Capacity curves of bolted IMCs for the MBSs.

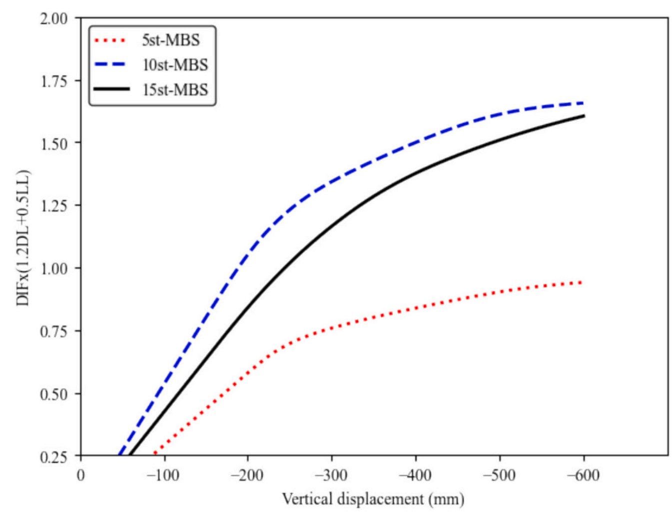


Fig. 13. Capacity curves of post-tensioned IMCs for the MBSs.

phase of the collapse scenario and provides valuable information about the behaviour of the structure under load. The first stage, the initial stage of the pushdown curve, where the model is stable, and all bolted and post-tensioned IMCs are in effect. At the second stage, as more weight is added, connection fails and the others follow, until they all fail one-by-one. This stage is critical as it determines the order that connections fail, which changes the curve's slope and can lead to collapse. However, in the scenarios studied, no connection failure occurred. The maximum dynamic increase factors for the examined MBSs with both types of connections were just under 2.0, which is slightly lower than the guideline-based DIF. As a result, the connections did not fail, indicating that structures should be designed to withstand at least twice the static service forces under dead loads, in addition to 50 % of the live load to prevent progressive collapse, in accordance with the guidelines [30,31]. During the final stage, typically the modules situated above the removed column will begin to fall (by failing connections one-by-one), ultimately leading to a structural failure. Once the structure has lost its stability and collapsed, the pushdown curve will level off, and the DIF associated with this stage is referred to as the 'progressive collapse capacity' of the building. This capacity represents the maximum load that the structure can bear before it collapses. It is essential to comprehend the behaviour of the pushdown curve at each stage to create structures that can resist a progressive collapse.

5.3.1. Bolted IMCs robustness assessment

To provide a clear basis for comparison, the capacity curves of modular buildings with 5, 10, and 15 stories with bolted IMCs have been plotted in Fig. 12. These curves depict the ability of the buildings to support the loads that were designed for in a scenario where the external column at the ground level is removed. By examining the curves for each building height, the differences in performance can be easily observed and analysed. In Fig. 12, there is a graphical representation of the capacity curves of 5-, 10-, and 15-story MBSs with bolted IMCs.

The research findings indicate that 10- and 15-story MBSs can withstand a failure load of 1.68 times the combined dead load and live load (1.2DL + 0.5LL), which is slightly below the safety threshold of 2.0, as outlined in current guidelines. This demonstrates that medium- and high-rise MBSs exhibit greater stability and structural integrity under gravity load conditions compared to their low-rise counterparts. The increased stability observed in the taller MBSs is attributed to the greater redundancy of their structural elements, which enhances their resilience during scenarios involving column removal. This observation is consistent with previous studies [22,27]. Furthermore, medium- and high-rise MBSs possess the highest initial stiffness and exhibit superior resistance to progressive collapse. In contrast, 5-story MBSs show a significantly lower DIF of 0.93 for bolted IMCs compared to medium- and high-rise MBSs. This substantial difference in DIF values indicates that low-rise MBSs with bolted IMCs are markedly less robust than their medium- and high-rise counterparts.

5.3.2. Post-tensioned IMCs robustness assessment

The capacity curves of post-tensioned IMCs for low- to high-rise MBSs are shown in Fig. 13. Nonlinear static pushdown analysis was performed to determine the failure load of post-tensioned connections for a 10-story MBS, which was found to be 1.66(1.2DL + 0.5LL), the highest DIF value. This value is nearly equivalent to that of bolted connections. Furthermore, the 15-story MBS with post-tensioned connections has a higher DIF value of 1.6 compared to the low-rise MBS, indicating lower robustness in comparison to the 10-story MBS with post-tensioned IMCs. The 5-story MBS with post-tensioned connections has a significantly lower DIF value of 0.94, similar to that of bolted IMCs. Overall, the 10-story MBS has the highest initial stiffness and progressive collapse-resisting capacity, followed by the 15-story and 5-story MBSs with post-tensioned IMCs, respectively.

MBSs can vary in height and use different types of connections to maintain their structural integrity. By conducting nonlinear static pushdown analysis on MBSs with bolted and post-tensioned IMCs, it was found that mid- to high-rise MBSs using both types of connections are more robust compared to low-rise MBSs since the taller structures will have more alternative load paths with the loss of the base story columns under load redistribution of progressive collapse. Moreover, the analysis indicates that mid- and high-rise MBSs with bolted connections demonstrate greater robustness compared to their post-tensioned counterparts, whereas low-rise MBSs with bolted connections show slightly lower robustness than those with post-tensioned connections.

5.4. Probabilistic demand models subjected to progressive collapse

The development of probabilistic demand models, as discussed in Section 2, provides an effective method for evaluating the probability of collapse in structures under progressive collapse scenarios with specific building specifications. These models determine the likelihood of reaching or exceeding a particular damage state based on input intensity variables.

After conducting nonlinear static pushdown analysis under progressive collapse scenarios, this study utilises probabilistic demand models to link EDPs to IMs as attributed to vertical displacement and load factor, respectively. The literature shows that the load factor is a strong predictor of structural responses under column loss scenarios for redistribution forces. Consequently, the conditional probability of failure is determined using this parameter as the IM.

Engineering demand parameters selection is critical since it determines structures' elastic and inelastic states [68,69]. The maximum vertical displacement is the most significant EDP in the evaluation of the fragility of frame structures subjected to column loss scenario. The safety of a building is a crucial aspect to consider, and it is determined by two limit states: building collapse and significant damage. The collapse limit state is defined following the GSA acceptance guideline, which specifies the criteria for the building's structural integrity. The significant damage limit state, on the other hand, is defined based on previous studies [70,71] that have shown when buildings become unsafe for occupants due to extreme nonlinearity. The ratio of peak vertical displacement-to-beam span is used to determine the limit states. For the significant damage limit state, a ratio of 0.5 % is adopted. For the collapse limit state, a ratio of 10.5 % is adopted. When the peak vertical displacement exceeds this value, the building is at risk of collapsing.

To compare low- to high-rise MBSs, a graph in Fig. 14 illustrates the correlation between VD as EDP plotted against DIF as IM in logarithmic scale. The results of the probabilistic demand model quantify the components of the studied MBSs with both types of IMCs. It is evident that there is a strong correlation between EDP and IM, with R^2 values falling within the range of 0.98 to 0.92.

The fragility functions depicted in Fig. 15 use both limit states of DIF under progressive collapse scenario for low- to high-rise MBSs with two studied IMCs. The results indicated that low-rise MBSs have the highest probability of exceeding both damage states for a given load, as seen in Section 5.3. Specifically, mid- and high-rise MBSs with bolted IMCs show similar and smaller exceedance probabilities than the low-rise counterparts. Likewise, low-rise MBS with post-tensioned IMCs has the highest probability of exceeding both significant and collapse limit state load factor compared to mid- and low-rise MBSs. In contrast to bolted IMCs, 10-story MBSs with post-tensioned IMCs possess the lowest probability of failure compared to 15-story and 5-story MBSs for both limit states.

The overall conclusion from the analyses is that the probability of failure of MBSs with different IMCs and building heights is not the same. MBSs with post-tensioned IMCs pose a greater risk of collapse due to low integrity than MBSs with bolted IMCs. Conversely, MBSs with bolted IMCs considerably reduce the potential for progressive collapse. In addition, low-rise MBSs are more likely to experience failure compared to mid- and high-rise MBSs for significant and collapse limit state under progressive collapse scenarios.

6. Concluding remarks

This study was conducted to explore the progressive collapse robustness of steel modular building systems (MBSs) with bolted and post-tensioned rod inter-module connections and different building heights. The research aimed to derive fragility functions for these buildings in a probabilistic framework. The first step was to conduct a nonlinear static pushdown analysis at varying levels of gravity loads. The analysis helped to investigate the global deformation of the MBSs based on the observed failure modes for the considered connections. High-fidelity nonlinear FE models were developedwith distributed plasticity elements to an external column loss scenario. This step was crucial because it helped to assess the collapse resisting capacity, ultimate failure load, collapse mechanism, and failure mode of the models. The nonlinear static pushdown analysis was then performed as a bay pushdown method to derive probabilistic demand models for two limit states of collapse and significant damage using the fragility method. In this study, fragility functions are developed, which provide a probabilistic framework for assessing the likelihood of structural failure, as well as a quantifiable measure of collapse risk, which can be used to inform modular structural design, safety standards, and mitigation strategies.

Considering structural response variability caused by different connection types and building heights, MBS can be evaluated more

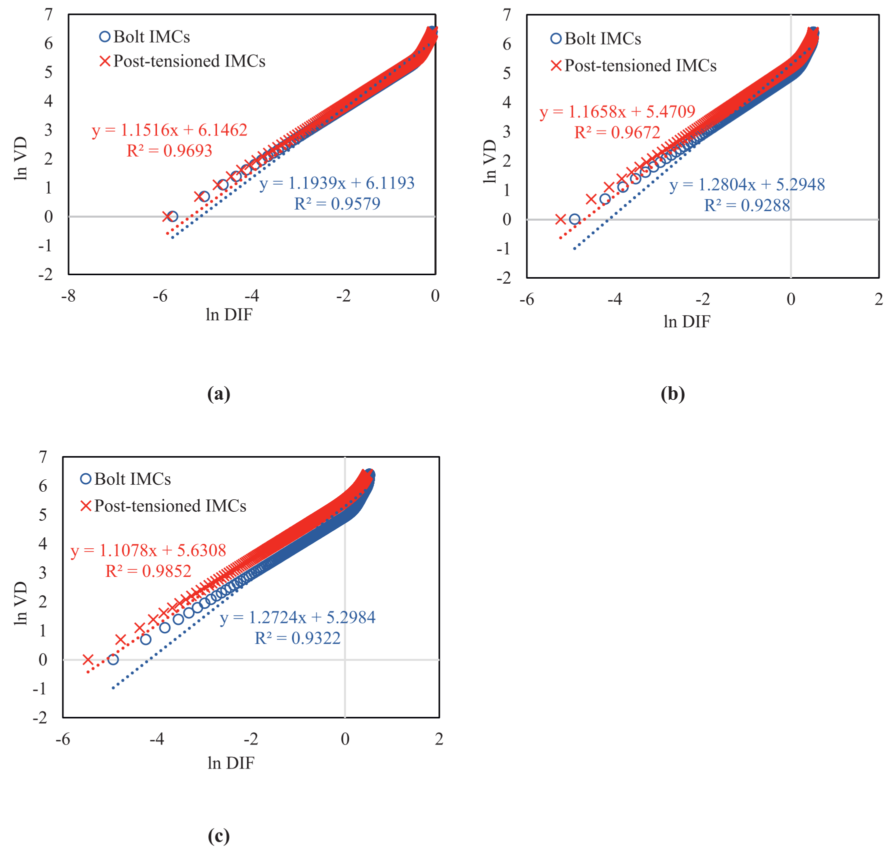


Fig. 14. Regression of the probabilistic demand model for (a) 5-story MBS, (b) 10-story MBS, and (c) 15-story MBS.

comprehensively. Despite the valuable insights that this study provides on the robustness of modular steel buildings with different IMCs and heights, additional scenarios need to be considered to provide more comprehensive results, including different removals of columns and modules, the number of modules per floor, and lateral resistance systems. The main conclusions are drawn as follows:

- The results from the nonlinear static pushdown analysis using the alternative load path method showed that even though there was a considerable redistribution in forces, there was no IMC connection failure in either the horizontal or vertical direction. Although the dynamic increase factors may not go over the DIF limit in GSA and DoD design guidelines, all building models remained stable and were robust enough to withstand the loss of a column. That is, the proposed DIF of 2.0 must be adopted based on the guidelines for designing the structure under column removal scenario to assess the progressive collapse capacity of the buildings.
- The measurement of the maximum lateral inter-story drift ratio of modular buildings under progressive collapse scenario was recorded and found to be below 0.05 %, which is an exceptionally small value when compared to the lateral limit state definition of buildings under seismic loading.
- IMC forces have been found to influence the axial and shear behaviour of vertical and horizontal connections in MBSs during progressive collapse. Several factors, such as the height of the building and the type of IMC used, influence this behaviour. Bolted IMCs tend to experience higher axial and shear loads during column loss compared to post-tensioned IMCs. For the axial behaviour of vertical IMCs, both types of connections experience the same forces. However, in the shear direction, different forces are applied to bolted IMCs as compared to post-tensioned IMCs. With regards to the horizontal behaviour of the IMCs, it has come to light that despite the discrete modelling of the floor modules, all floors across all levels of the building operate as rigid diaphragms. This finding underscores the adequate stiffness of the horizontal IMCs to maintain such behaviour. In addition, the zigzag pattern observed in the peak force of the horizontal IMCs along the height of the building demonstrates that modules located above the damaged ones exhibit incomplete cantilever action, indicating an inability to effectively redistribute forces under the removal of the column.
- Robustness assessment was carried out for various MBSs that had different connections. The results illustrated that mid- and high-rise MBSs exhibit greater robustness when compared to low-rise MBSs for both types of IMCs due to the greater redundancy of their structural elements, specifically the presence of double beams and columns, which allow these structures to better distribute forces. Moreover, the study found that 10- and 15-story MBSs with bolted IMCs demonstrated higher levels of robustness when compared to the other studied MBSs. On the other hand, 10-story MBSs with post-tensioned IMCs were found to be more robust than their 15-story counterparts. In general, mid- and high-rise MBSs with bolted connections exhibit greater robustness than their post-tensioned counterparts, while low-rise MBSs with bolted connections display slightly lower robustness compared to those with post-tensioned connections.
- At last, the fragility of various MBSs with different building heights and connections was studied using fragility functions. Two different limit states were taken into consideration for the analysis. The results indicated that the probability of failure for mid- and high-rise MBSs with bolted IMCs across the studied limit states is lower than for their post-tensioned IMC counterparts, while the probability of failure for low-rise MBSs is almost similar for both connection types. Moreover, the study revealed that low-rise MBSs are more likely to exceed both

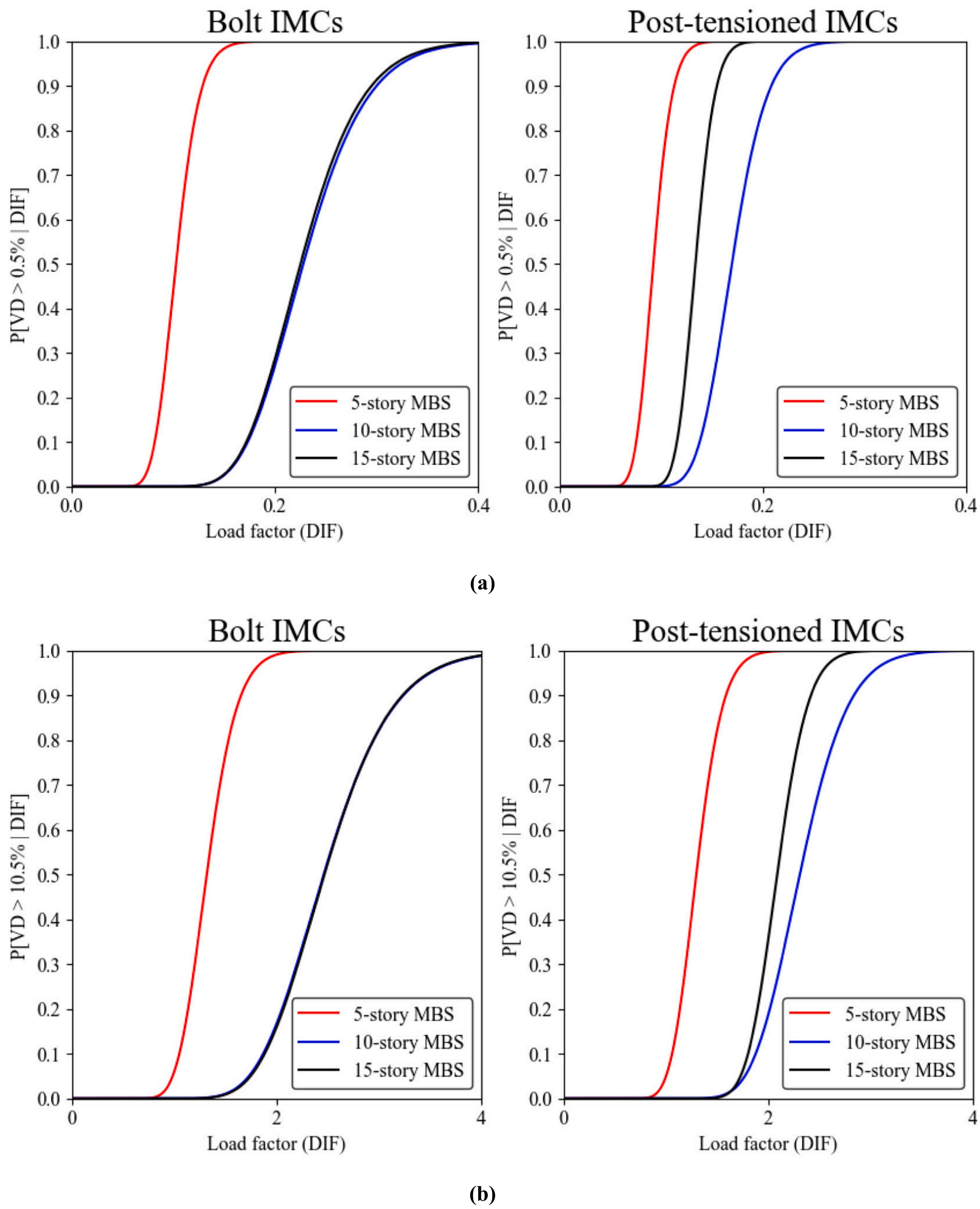


Fig. 15. Probabilistic demand models of MBSs for bolted and post-tensioned IMCs under progressive collapse scenario for (a) Significant limit state, and (b) Collapse limit state.

damage states for the studies scenario compared to mid- and high-rise MBSs.

CRediT authorship contribution statement

Amirhossein Emamikoupaei: Writing – original draft, Visualization, Methodology, Investigation, Formal analysis, Data curation. **Konstantinos Daniel Tsavdaridis:** Writing – review & editing, Validation, Supervision, Project administration, Methodology, Investigation, Conceptualization. **Ali Bigdeli:** Data curation, Formal analysis, Methodology, Software, Validation. **Kimia Saffarzadeh:** Data curation, Resources, Software.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

No data was used for the research described in the article.

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