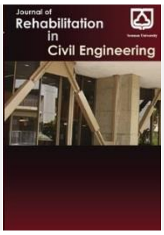

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The Influence of Inter-Connections Characteristics on the Lateral Performance of Braced-Frame Modular Steel Buildings

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ABSTRACT

The use of pre-fabricated modular units for the construction of mid to high-rise buildings has been promoted recently. The modular units are fabricated in a factory and then transported to the construction site to form a structure using inter-connections. The inter-connection is unique to modular buildings, playing a critical role in their structural performance. Despite the increasing popularity of modular construction, there are relatively few published studies considering the influence of inter-connection's behaviour on the lateral performance of braced-frame modular buildings. The inter-connections' rigidity has an influence on the modular buildings' stiffness. Hence, it is required to investigate their effect on the seismic performance of braced-frame modular buildings. This study aims to investigate the effect of inter-connections' properties on the lateral performance of braced modular frames through nonlinear static analysis. To that end, three frames of four, eight and twelve storeys, are assumed for the required analysis. Different inter-connections having various stiffness properties are considered for the nonlinear static analysis. Three performance levels are considered and the responses of considered structures at these levels are evaluated and compared for different properties of inter-connections. The obtained responses indicate that the decrease of inter-connections' stiffness leads to reduction of the lateral capacity. The results indicate that the decrease of inter-connection stiffness can increase the period of the structures up to 10.35%., 5.35% and 3.63% in 4-, 8- and 12-storey buildings. Moreover, the nonlinear analysis indicate that the increase of inter-connection flexibility reduces the base shear by 1.9%.

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1. Introduction

Modular construction is a term employed to describe the use of factory-produced large volumetric building units that are delivered to the site of a building and assembled together, through the use of novel connections, called inter-modular connections or inter-connections, to form a complete building**.** The construction of buildings using modular construction is not a new concept. However, it is greatly revitalised by recent technological advances, allowing to build of a variety of high-quality modular units for high-performance buildings' manufacturing, which is the leading edge of research in building construction. The modular construction method provides several advantages compared to the conventional on-site steel structures such as quality improved, onsite construction time and human power saving, reduce material wastage, and potential for reusability [1]. However, despite providing such advantages, as modular construction components are prefabricated and transported to the site of the building, there are some damage risks during transportation. Moreover, few studies have been conducted on the performance of these structures under gravity and lateral load. Inter-connections, as well as the connection of beams to columns, which are known as intra-connections, are key components of these structures. The integrity of modular buildings significantly relies on the mechanical behaviour of joints, including intra- and inter-connections, which provide pathways to transfer the gravity and lateral loads [2–5]. The use of inter-connections in modular buildings alters the boundary condition at both ends of columns.

Compared to a joint in conventional steel structures, which is made of up to four beams and two columns, a joint in corner-supported modular buildings is comprised of up to eight columns and sixteen beams which are connected together through a combination of inter- and intra-connections. This results in an increase in the complexity of these buildings' responses to both gravity and lateral loads. Fig. 1 shows two modular units which are connected together through the use of inter-connection.

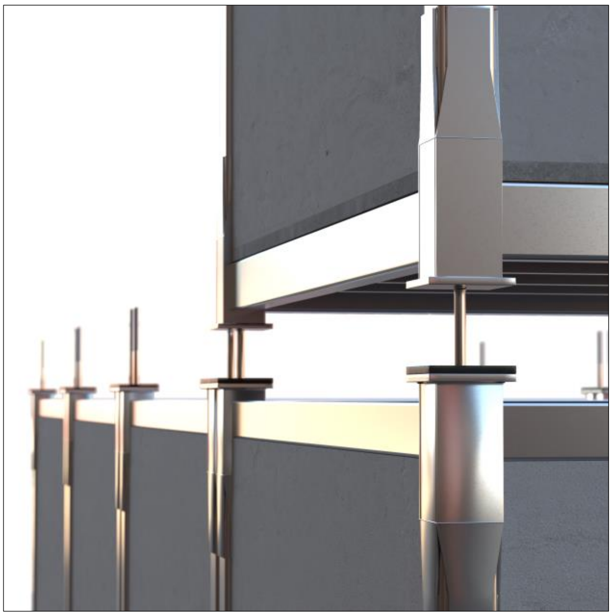

Fig. 1. Two modular units connected with inter-connections.

Due to such complex boundary conditions, the nonlinear and linear lateral performance of Corner-Supported Modular Steel Buildings (CSMSBs) may differ from those of conventional buildings when subjected to lateral loads including earthquake and wind loading. Different configurations, mechanical behaviour of joints and detailing requirements may affect CSMSBs' lateral stiffness [6–10], dynamic characteristics , and seismic behaviour [9,11,12]. For example, Annan et al. [13,14] observed that the reserve strength of braced frame CSMSB is greater than that prescribed by the Canadian code for regular braced systems. Moreover, the analysis conducted by Annan et al. [14] showed that these structures possess a significant ductility capacity. Farajian et al. [3] conducted a comprehensive numerical simulation to quantify the seismic performance factors of braced frame CSMSB through a comprehensive nonlinear static and time history dynamic analysis. They concluded that these systems have a higher *R-factor* and overstrength factor compared to performance factors suggested by available codes for the design of conventional steel structures. Despite the fact that several inter-connections have been developed to link the modular units together (e.g. [15–20], limited published data is available on the influence of inter-connections' mechanical properties on the nonlinear performance of CSMSBs. The limited available data on the lateral performance and seismic behaviour of modular structures hinders the development of design guidelines, standards and codes. As a result, these structures are being designed based on the available design guidelines and codes for conventional buildings, in which engineers need to take the actual behaviour of inter-connections into account to accurately model and design of CSMSBs.

Some research has been conducted to determine the influence of inter-module connections on the lateral performance of corner-supported modular steel structures. In 2021, Lacey et al. [21] proposed new simplified inter-module connections to investigate the effect of the inter-module connections on the overall responses to wind and earthquake loading. Their study proposed inter-storey drift ratio limits based on simplified connection behaviours. According to the results, the new simplified models were well suited for use in global numerical simulations. In the other study, Lacey et al. [8] examined the effect of inter-module connection stiffness on the structural response of a six-story modular steel building subjected to wind and earthquake load. The results showed that the translation stiffness of inter-connections significantly affects the overall response of the considered modular structure. The effects of inter-module connection modelling on the global sway behaviour of high-rise modular buildings with different lateral force-resisting systems were examined by Chua et al. [22]. A more realistic approach of modelling the floor slab consisting of multiple modules inter-connected at the corners was recommended, and corner-connected modules were evaluated for their effectiveness in transferring horizontal forces to the building's lateral load resisting systems. In a study by Peng et al. [23] tenon-connected inter-module connections were used to investigate the lateral resistance of multi-storey modular buildings. Timoshenko beam elements and spring elements were used to create a simplified joint model. Their results showed that a simplified joint model is capable of accurately predicting the damage evolution of the tenon-connected inter-module connection, including bolt slippage, weld fractures, and joint distortions.

This paper, which is part of a broad project on the compliance criteria for the design of inter-connections in corner-supported modular structures aims to study the influence of inter-connections' properties on the nonlinear behaviour of corner-supported braced modular buildings. To that end, three braced corner-supported modular frames having a different number of storeys (4, 8 and 12 storeys) are considered for the required nonlinear analysis. The considered structures are designed based on the available design guidelines. Then, the designed buildings are subjected to pushover analysis to determine their lateral performance and study the influence of inter-connections' mechanical properties on the lateral performance of corner-supported modular structures. Three performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) in accordance with FEMA 356 are considered and the lateral performance of modular structures with different behaviours of inter-connections are compared together at these performance levels.

2. Design assumptions assumptions and description of the model

2.1. Structural design

To conduct the required analysis, three CSMSBs having different storey heights are used as case studies. The structures include a low-rise structure (4 storeys), a mid-rise structure (8 storeys), and a high-rise structure (12 storeys). Figure 2 (a) illustrates the 3-dimension view of the considered modular building with four storeys. The buildings considered for this study are all used for residential purposes. There are twenty-five modules in each storey, including five modules in each direction. The modules include both ceilings and floors; their dimensions are 6.1 meters in X, 3.6 meters in Y, and 3.2 meters in the Z direction. Mechanical, electrical and services can run between modules by allowing a gap of 0.2 m between each module in the X, Y, and Z directions. Therefore, the plan dimensions of the structures are 31.3x18.8 m, and their heights are 13.2 m, 27 m, and 40.6 m. The plan view of the structures is depicted in Figure 2 (b).

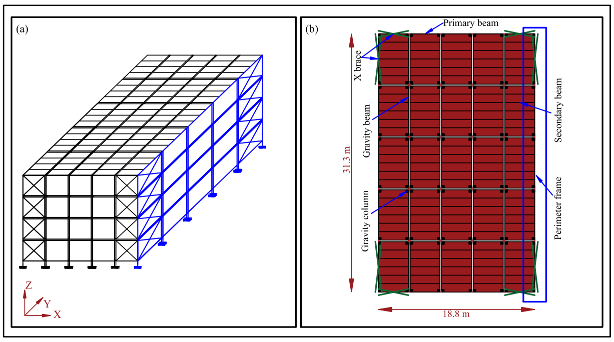

Fig. 2. (a) 3-dimension view of the four-story building (b) plan view of the considered building.

The bondeck system is considered for the floor system, which has a slab thickness of 150 mm, a compressive strength of $400*10^5$ N/m^2, and a weight per unit volume of γ=23563 N/m^3. The Hebel system, which utilizes lightweight concrete, is employed as a ceiling, with γ= 6520 N/m^3, a compressive strength (f'_c) of $400*10^5$ N/m^2, and a slab thickness of 200 mm. Table 1 provides the super dead load utilized for additional loads, including the floor and ceiling, as well as the live loads used for the design.

Buildings are constructed by linking modules together via Vertical and Horizontal Inter-connections (VCs and HCs), described in the following section. A symmetrically distributed set of X-braced bays provides resistance against lateral loads in each direction as shown in Figure 2 (b), which are positioned at the corners of the structure's plan. To simplify the mathematical modelling of structures, some parts and aspects of a building's configuration are not taken into account in the 3D models, such as infill walls, façades, and staircases.

Table 1. Applied loads to the studied model.

Structural components	Load type	Load
Floor slab	Super dead load	1000 N/m^2
	Live load	192 N/m^2
Ceiling slab	Superimposed dead load	200 N/m^2
	Live load	50 N/m^2

The considered buildings are designed based on ASCE/SEI 7-16 [24] for lateral load loading and AISC 360-16 for steel structure design. The spectral response acceleration parameters for periods of 1s (S_1) and short periods (S_s) are selected, assuming that the buildings are constructed on a site, which is classified as a seismic design category (SDC) of D_{min} based on ASCE/SEI 7-16 and FEMA P695 [25]. Structures are designed against earthquakes using the Equivalent Lateral Force method (ELF). The seismic performance factors for “Dual systems with intermediate moment frames” were selected as a preliminary design factor due to a lack of data.

The 3-dimension model of buildings is developed using the finite element software SAP2000. This finite element software has the capability to model a wide variety of elements including frame and link elements with linear or nonlinear behaviours. Inter-connections are simulated using the linear behaviour of link elements. The linear behaviour can be justified by the fact that connections are critical components of a structure and therefore, any damage to these critical elements during an earthquake is not feasible or acceptable. In the mathematical model of buildings, horizontal and vertical inter-connections are modelled separately. Gusset plates are welded to brace elements to connect them to the modular framing system which are connected using pinned connections without having moment transfer capability to their adjacent beams and columns. On the other hand, a rigid behaviour is assigned to the intra-modular connections. Moreover, it is assumed that columns are fully fixed to the foundations, therefore the rotation degree-of-freedoms about X, Y, and Z directions are restrained in SAP2000. The geometrical nonlinearity is considered through the P-Δ effects in SAP2000. Furthermore, beams, columns, and brace elements are designed using square hollow structural sections (HSS), which meet seismically compact criteria as suggested by AISC 360-16 [26]. For practical purposes, it is assumed that the columns' sections are the same in each storey. As a result, some elements may be conservatively designed. As illustrated in Figure 2 (b), the perimeter frame is used to conduct the required analysis. Figure 3 (a) to (c) illustrate the details of the designed structures.

2.2. Inter-connections

A modular structure uses VCs and HCs to link modules at their corners. This allows them to resist both gravity and lateral loads together. In this study, the horizontal and vertical connections from the works conducted by Styles et al. [7] are adopted to connect modular units together. Based on their work endplate connections are used to connect modular units. Detailed models of envisaged connections were first developed in their study to determine the axial, shear, and rotational stiffness required for a modular building located in Australia. Styles et. al employed a typical double-cleat angle connection as the intra-connection.

The connection geometry was detailed as per section 4 and Appendix G of Joints in Steel Connections: Simple Connection (JSC) design guide.

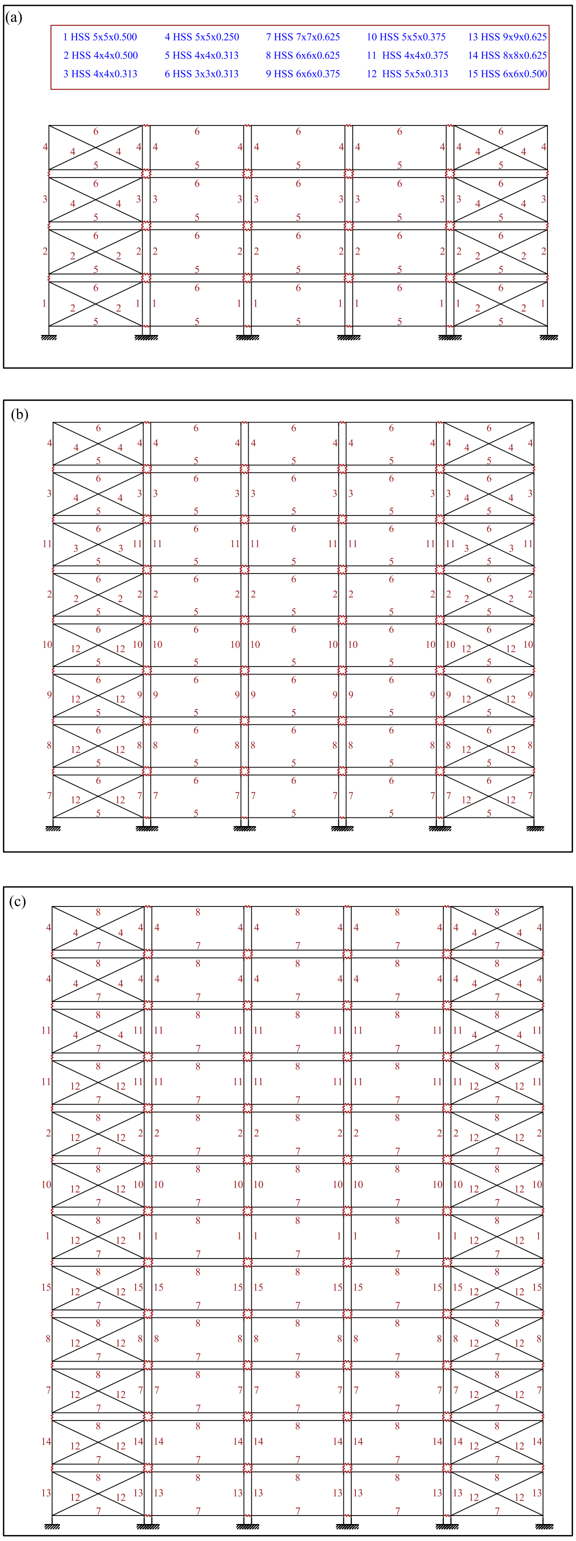

Fig. 3. Design details of (a) four-storeys (b) eight-storeys (c) twelve-storeys.

On the other hand, they used a typical column splice connection end plated welded to the column end to connect modular units vertically and side plates with bolts and nuts to connect modular units horizontally. The connection details are described in detail in [7]. In their study, they performed finite element simulations to extract the force-displacement, moment-rotation and therefore, the rigidity of HCs and VCs. Then, they examined the influence of the rotational stiffness of joint on the structural responses of a multi-storey modular building. Figure 4 shows the schematic horizontal and vertical inter-connections used by Styles et al.

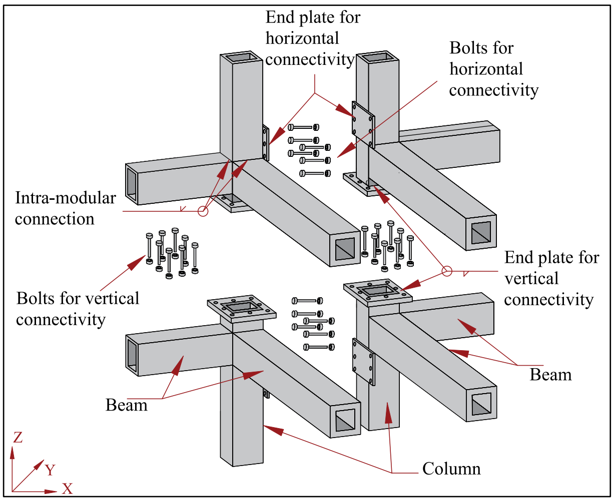

Fig. 4. Detail of horizontal and vertical inter-connection used by Styles et al. [7].

The numerical analysis published by Styles et al. includes the force-displacements of HC in Z and X directions, as well as moment-rotations about Y and X directions. For VC, they published force-displacement data in X, Y, and Z directions, but no data on moment-rotation was published. The force-displacement and moment-rotation of both HC and VC are depicted in Figures 5 (a) to(e). A line is fitted to the force-displacement of each component to determine their stiffness, which are used for the required analysis. Since there are insufficient data available and, in an effort to simplify the problem, it has been assumed that the horizontal inter-connection will behave in the same manner in both directions in X and Y. Furthermore, the presented moment-rotation behaviour of horizontal inter-connections about X, Y, and Z directions can also be applied to vertical inter-connections about X, Y, and Z.

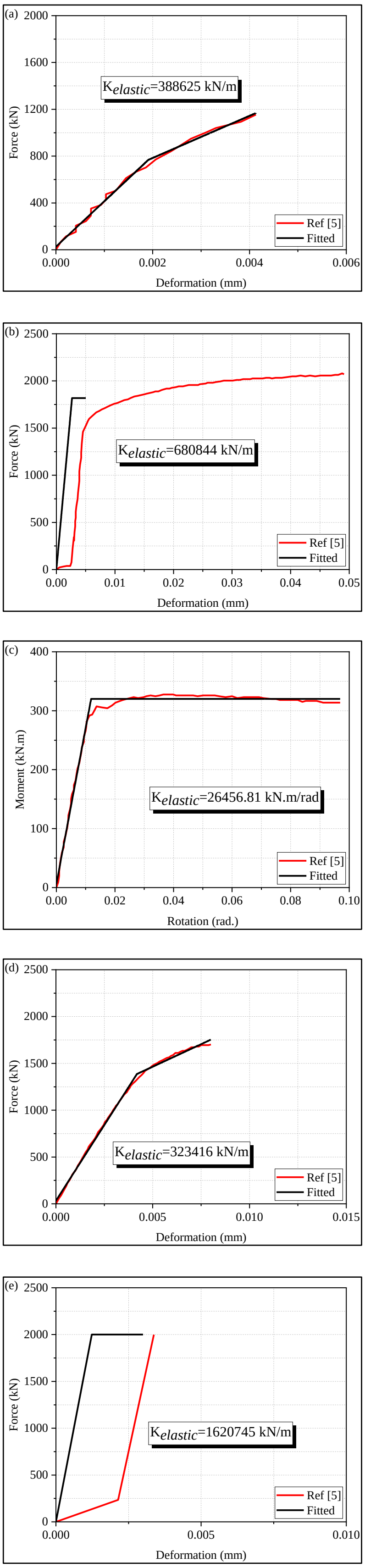

Fig. 5. The behaviour of (a) HC in the X direction (b) HC in the Y and Z directions (c) HC about the Y direction (d) VC in the Z direction (e) VC in the X and Y directions.

Figure 5 (a) indicates the axial force-displacement of the horizontal inter-connection, i.e. the connection is subjected to tension. The resulting behaviour can be attributed to the plate bending stiffness as well as bolt tension. Hence, the failure mode is a combination of tension failure of the bolts and yielding of the plate. Figure 5 (b) shows the shear behaviour of HC, i.e. the connection is subjected to shear. The force-displacement curve shows three stages. The first stage which has a low stiffness is as a result initial slip of the connection, followed by the second stage which is due to the contact of bolts with the edge of the bolt hole. The stiffness of the connection increases at this stage compared to the first stage. The third stage is the yielding and therefore, reduction of the stiffness in this stage. In order to model these behaviours, the first and third stages are ignored to simplify the model. Figure 5 (c) illustrates the M-θ behaviour of the connection, indicating an elastic stiffness, followed by stiffness reduction due to the yielding of the plate. For the VC, when the connection is subjected to shear, the force-displacement curve shows a two-stage bilinear behaviour. The first stage is due to the initial slip of the bolder connection, and the second stage, which is associated with an increase in stiffness, occurs when the bolts contact with the bolt hole edge.

The mechanical properties of inter-connections, particularly their stiffness, may affect the seismic performance of a modular structure. To better understand the effect of inter-connection properties on the seismic behaviour of modular steel buildings in both linear and nonlinear ranges, a parametric study is conducted. For the parametric study, various combinations of stiffness are considered for both VC and HC. According to the reference paper [7], stiffness properties are subjected to two flexibility factors, 0.7 and 0.5. To represent an ideal hinge connection, a flexibility factor of 0 is assigned to the rotational stiffness of inter-connections. A rigid behaviour is also included, in which the stiffness of different inter-connection components is assigned a very high value. Table 2 lists the names of the inter-connections and their stiffness properties.

Table 2. Names and properties of interconnections used for parametric analysis.

Model name	VC			HC			Flexibility factor	Component	Connection
	u_1 (N/m)	u_2 (N/m)	r_3 (N.m/rad)	u_1 (N/m)	u_2 (N/m)	r_3 (N.m/rad)			
IC1	1E+20	1E+20	1E+20	1E+20	1E+20	1E+20	-----	-----	-----
IC2	323416000	1620745543	26456810	388625000	680844000	26456810	-----	-----	-----
IC3	323416000	**1134521880**	**18519767**	**272037500**	**476590800**	**18519767**	0.7	Shear, axial and rotation	VC and HC
IC4	323416000	**810372771.5**	**13228405**	**194312500**	**340422000**	**13228405**	0.5		

2.3. Nonlinear modelling of structures

OpenSees framework is used for nonlinear static analyses. This finite element software can be used to model elements, springs, and materials, which can have a wide range of linear and nonlinear behaviours. A variety of analyses can also be performed with this open-source software. In order to conduct the analysis, a 2D model of each building is generated in OpenSees. Figure 6 (a) to (c) illustrates the details of the finite element model of frames.

The nonlinear static analysis is conducted by modelling beams and columns with the *forceBeamColumn* element which uses distributed plasticity. The geometrical nonlinearity is considered in the modelling of the structures by employing the P-Delta transformation command in OpenSees.

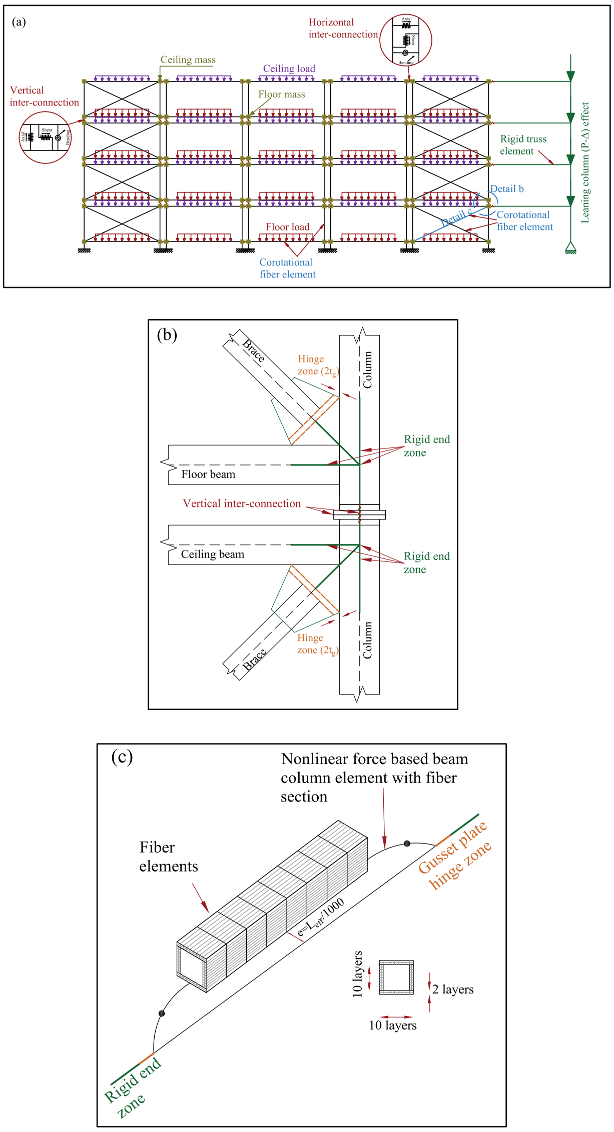

Fig. 6. (a) Detail of the finite element model for the frames in OpenSees, (b) detail of gusset plate connection, (c) detail of brace modelling with gusset plate.

Using gravity columns, additional lateral loads are induced to the lateral resisting frames through leaning columns. Nodes on leaning columns at different storeys are imposed with half the ceiling and floor weights, representing gravity loading. Nonlinear beam-column elements and fiber sections are used to simulate brace elements in OpenSees using displacement-based element types. By modelling distributed plasticity and numerical integrations over the length of the member as well as the cross-section of the member, the element considers nonlinearity. To ensure that brace elements can buckle during nonlinear static analyses, an initial imperfection equal to 0.001 effective length is imposed at the middle of the brace elements. In order to obtain a more accurate result, 10 segments are included in each brace element. By employing this modelling approach, it is possible to monitor and capture the post-buckling behaviour of brace elements. Through corotational transformation, geometrical nonlinearity is considered in braced elements. Steel materials are used in the elements, which are modelled using the uniaxial material type Steel02 from the OpenSees materials library. A strain hardening ratio of 1% is assumed for elements that exhibit behaviour beyond the elastic range. In this paper, the ‘twoNodeLink’ element available in OpenSEES is utilised to model vertical and horizontal inter-connections with semi-rigid behaviour. It is assumed that these connections are safeguarded by capacity limitations and therefore only their elastic behaviour is considered in the required analysis. For the initial structural design, a rigid behaviour is assigned to the twoNodeLink elements to make sure that both horizontal and vertical inter-connections are behaved rigidly. On the other hand, for required nonlinear static analysis, Table 2 is used to assign various components of multidirectional stiffness to horizontal and vertical inter-connections, therefore inter-connections have a semi-rigid behaviour. Figure 5 illustrates the stiffness of twoNodeLink elements in different directions based on the numerical data published by [7]. Each module’s rigid diaphragm in the ceiling and floor is simulated through the equalDOF command in OpenSees. For the purposes of the required analysis, storey masses at both the ceiling and the floors are represented by nodal masses. It is assumed that all beam to column connections in CSMSBs are moment-resisting due to the fact that in reality beams are fully welded to columns. The bases of columns are fully fixed to the ground, therefore, the columns are fixed about the rotations in all directions. As part of the nonlinear dynamic analysis, Rayleigh damping is used to account for damping by assigning a 5% damping ratio to the first and second modes of vibration. It is possible to encounter non-convergence problems during the analysis of nonlinear static. It is therefore necessary to define a solution algorithm object in order to ensure the accuracy of the numerical solution. A sequence of steps is identified in the algorithm for solving the nonlinear equations. Different algorithms are used to find a convergent solution in the case of non-convergence.

3. Nonlinear static pushover analysis

Performing a nonlinear static pushover analysis is primarily intended for controlling and verifying the inelastic performance of a frame as well as its lateral strength. Furthermore, it is also capable of capturing different mechanisms of failure that are likely to be seen in a nonlinear time history analysis. Despite the fact that static pushover analysis is an appealing method to evaluate the performance of structures, this method has some limitations. The primary limitation of this method is that it employs static analysis to capture dynamic effects, which in some cases, such as highrise buildings, may result some inaccuracies. During the procedure, the drift/displacement at a specific point is monotonically increased, which is distributed over the height of the frames until it reaches a predetermined value (target displacement/drift). In this procedure, the results are used to produce a relationship between top story displacement/drift and structural base shear, which is referred to as a capacity curve or pushover curve. Several approaches to lateral load distribution can result in pushover curves, each with its own characteristics and sequence of plastic hinge formation. It has been demonstrated by Mwafy and Elnashay [27] that the use of inverted triangular lateral load distribution, as suggested by ASCE/SEI 7-16, results in a more accurate assessment of maximum structure drift and R factor. Furthermore, multiple load patterns do not significantly improve the accuracy of data produced by nonlinear static analysis [27]. Accordingly, in this research, frames are preloaded with factored gravity combination loads, defined by Equation (1) [25], and then exposed to a statistically earthquake-derived triangular distribution to generate pushover curves.

$$1.05DL + 0.25LL \tag{1}$$

where DL and LL are dead and live loads, respectively.

FEMA 356 [28] proposes three performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for the performance evaluation of structures. According to FEMA 356, the drift of a storey in a structure corresponding to the IO performance level for braced frame structures is 0.5% transient and negligible permanent. On the other hand, the maximum drift of a storey in the considered structure corresponding to the LS performance level is 1.5% transient or 0.5% permanent. Finally, FEMA 356 offers a 2% transient or permanent storey drift in a building corresponding to the CP performance level. These performance levels are also adopted in this study to investigate the effect of inter-connections' mechanical properties on the storey shear and storey's displacement of considered buildings.

4. Results and discussion

This section provides information on the lateral performance of considered modular structures through performing a set of nonlinear static analyses on the assumed structures having inter-connections with different behaviours, tabulated in Table 2. The target drift is assumed to be 5% of the total height of the structure. The aim is to investigate the influence of inter-connections' stiffness properties on the lateral behaviour of considered structures. Figure 7 (a) to (c) illustrate the pushover curves of three considered structures, having different behaviour of inter-connections.

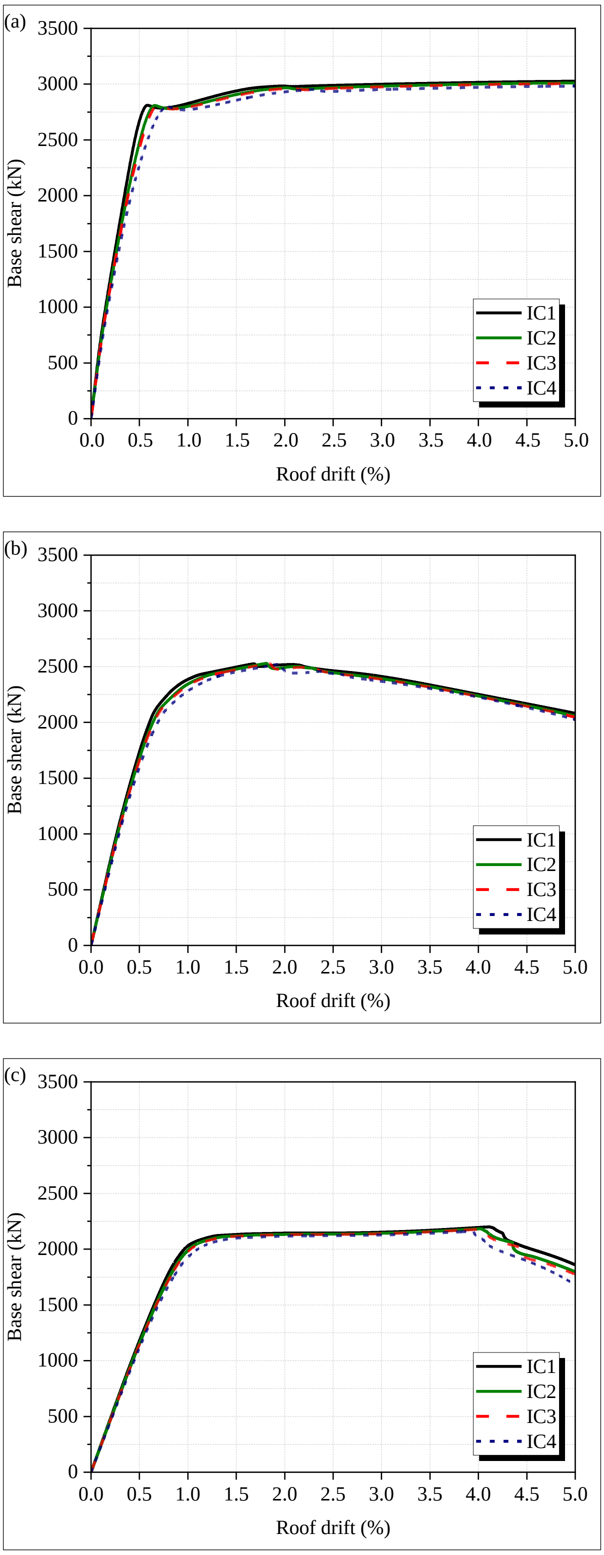

Fig. 7. Pushover curves of (a) four-storey, (b) eight-storey, and (c) twelve-storey buildings.

The graphs indicate that the inter-connections' stiffness has no significant effect on the linear behaviour of considered structures. The linear characteristics of a structure can be well represented by its period. Table 3 lists the first mode of structures and the fundamental period of structures determined from ASCE/SEI 7-16 for regular braced frame structures.

Table 3. Results of eigen analysis of each structure (in seconds).

Connection's name	Eigen analysis			ASCE/SEI 7-16 (= 0.0488 $h_n^{0.75}$)		
	4-storey	8-storey	12-storey	4-storey	8-storey	12-storey
IC1	0.483	1.308	2.392			
IC2	0.504	1.336	2.425	0.342	0.578	0.785
IC3	0.510	1.344	2.436			
IC4	0.533	1.378	2.479			

The eigen analyses show that decreasing the inter-connections' stiffness leads to an increase in structures' period. This is more evident in the 4-storey structure. In this structure, the use of IC2, IC3, and IC4 inter-connections results in 4.35%, 5.59%, and 10.35% increases of its first mode. With reference to the 8-storey modular building, changing the behaviour of inter-connection from rigid behaviour (i.e., IC1) to the most flexible inter-connection (i.e., IC4) leads to an increase of almost 5.35%. A similar trend was observed for the 12-storey building. While the fundamental period of the structure with rigid behaviour of inter-connections was about 2.392 seconds, it reaches to 2.479 seconds when the IC5 inter-connection is used in the structure, a 3.63% increase in the first period of the structure. This shows that the stiffness properties of inter-connection have no significant effect on the behaviour of structures in the linear range. Based on eigen analysis, CSMSBs' fundamental periods are underestimated compared to those derived from ASCE/SEI 7-16. Lacey et al. [8] and Sanches et al. [29]

have also reported the underestimation of fundamental period determination obtained from standard codes for conventional buildings compared to the one obtained from eigen analysis.

According to Figure 7 (a), the pushover curve of the 4-storey structure with IC1 inter-connections shows an elastic behaviour up to 0.53% roof drift level, followed by a sudden stiffness reduction up to the peak lateral capacity of 3026 kN at 2.74% roof drift level. The lateral capacity of the structure remains almost constant up to the roof drift of 5.0%. In the case of the 4-storey structure with IC2 inter-connections, the structure has a linear behaviour up to 0.57% roof drift level. Following a sudden reduction in the lateral stiffness of the structure, its peak lateral capacity occurs at 3% roof drift and is 3010 kN. It remains almost constant up to the roof drift level of 5.0%. Employing the most flexible inter-connection (IC4) shows an almost similar trend in the pushover curve. The structure behaves linearly up to 0.63%, followed by an abrupt stiffness reduction. The lateral capacity reaches its maximum, 2982 kN, which indicates a 1.4% decrease compared to the IC1, and 1% compared to IC2. This illustrates that the stiffness properties of inter-connections have an insignificant effect on the lateral capacity of the structure.

With reference to Figure 7 (b), the structure has a linear behaviour up to 0.6%, 0.63%, 0.65%, and 0.67% corresponding to IC1, IC2, IC3, and IC4, respectively. The lateral capacity increases gradually to its maximum in all cases at the roof drift level of 1.68%, 1.81%, 1.85%, and 1.95%, corresponding to IC1, IC2, IC3, and IC4, respectively. The maximum base shear of the structure is 2525 kN, 2528 kN, 2526 kN, and 2523 kN, respectively. This shows that the use of different behaviours of inter-connection has no significant effect on the lateral capacity of the structure. Following that, the lateral capacity of the 8-storey structure with IC1, IC2, and IC3 inter-connections slightly decreases up to 5% roof drift level. This is mainly due to the P-Δ effect in this structure.

The pushover curve of the 12-storey structure with rigid behaviour indicates a linear behaviour up to 0.83% roof drift level, in which the base shear reaches to 1832 kN. The pushover curve of the structure with IC4 shows a linear behaviour up to 0.81% of roof drift level, where the base shear reaches to 1688 kN. The lateral capacity of structure with rigid inter-connection increases up to 2200 kN. Employing IC4 inter-connection, the base shear increases to 2159 kN, indicating a 1.9% reduction in the maximum base shear. Beyond the roof drift level of 4.25%, the P-Δ affects the lateral capacity of the structure, in which a drop in the push curve is observed. In fact, the instability effects caused by p-delta actions need to be taken into account in interpreting the various pushover curves of structures. While in the short shear-dominated four-storey structure, the effect of P-Δ results in the localised amplification of storey drifts, in the tall structures, including eight- and twelve-storey buildings amplification of the overall overturning moment is responsible for the P-Δ instability, leading to a drop in the capacity curve of the buildings. The observed lateral performance of considered structures indicates that the behaviour of inter-connections has no significant effect on their behaviour in both elastic and inelastic ranges.

Figure 8 (a) to (c) show the inter-storey drift of the 4-storey modular building corresponding to the IO, LS, and CP performance levels. The figure illustrates that the maximum difference between the inter-storey drift of IC1 and IC4 are 9.2%, 9.5%, and 10.1% corresponding to IO, LS, and CP performance levels. This shows that the stiffness of inter-connections has no significant effect on the inter-storey drift of the building.

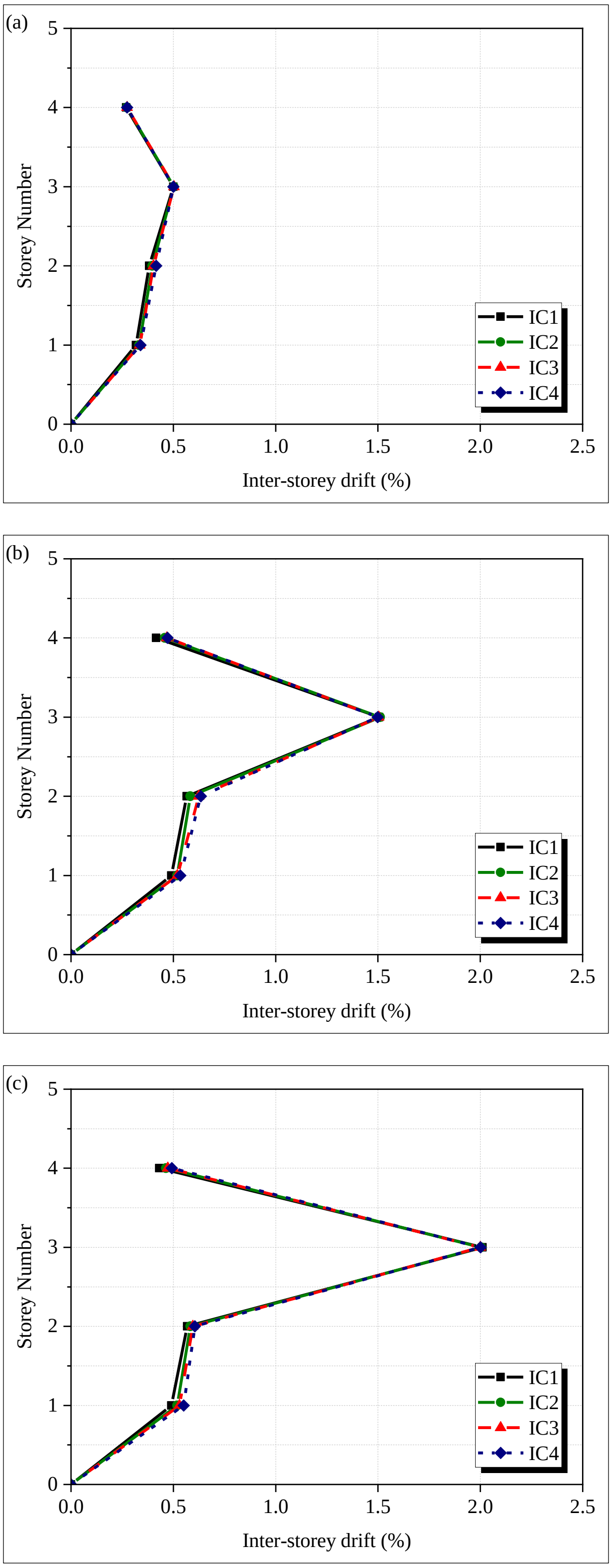

Fig. 8. Inter-storey drift ratio of the 4-storey building (a) IO (b) LS (c) CP performance levels.

Figure 9 (a) to (c) indicate the inter-storey drift of the 8-storey designed building corresponding to the IO, LS, and CP performance levels. The obtained results show that the maximum difference between the inter-storey drift of the building corresponding to the stiffest and most flexible inter-connections (i.e., IC1 and IC4) are 5.3%, 7.3%, and 9.1%, respectively.

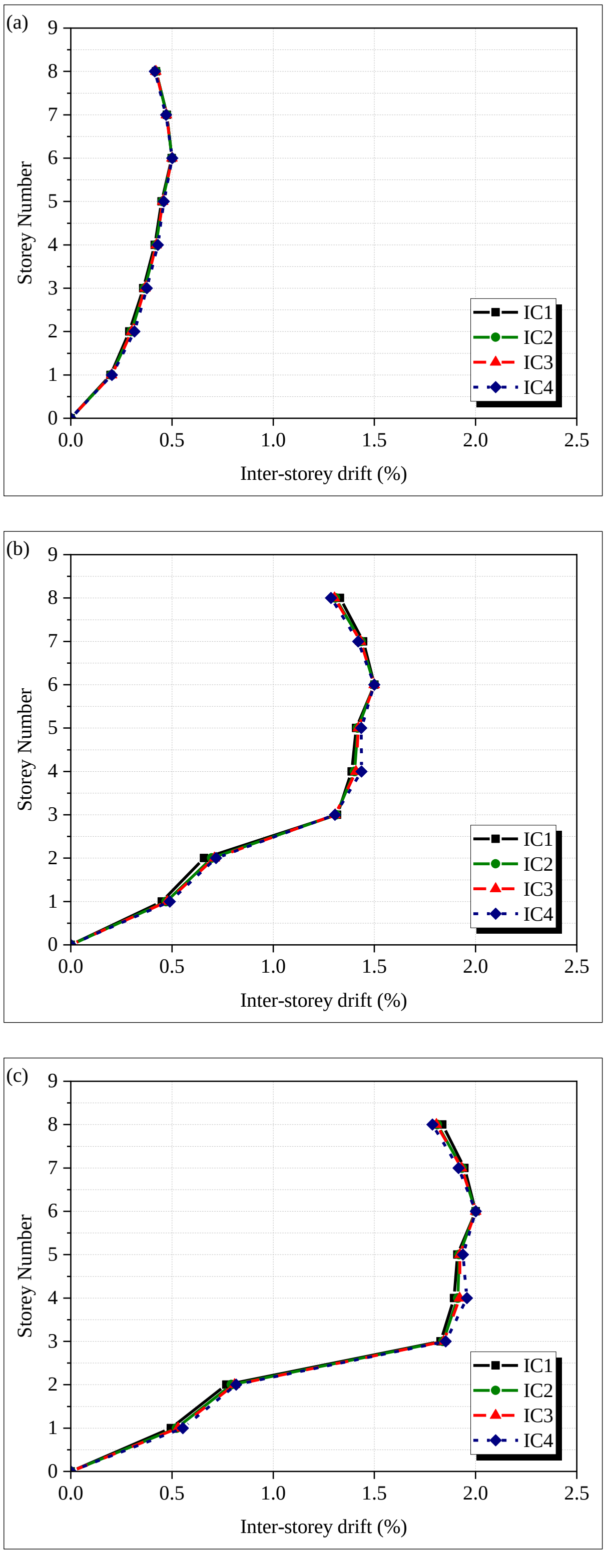

Fig. 9. Inter-storey drift ratio of the 8-storey building (a) IO (b) LS (c) CP performance levels.

The inter-storey drift of the 12-storey building corresponding to IO, LS, and CP performance levels are shown in Figure 10 (a) to (c), respectively, showing a similar trend compared to 4- and 8-storey can be observed for the inter-storey drift.

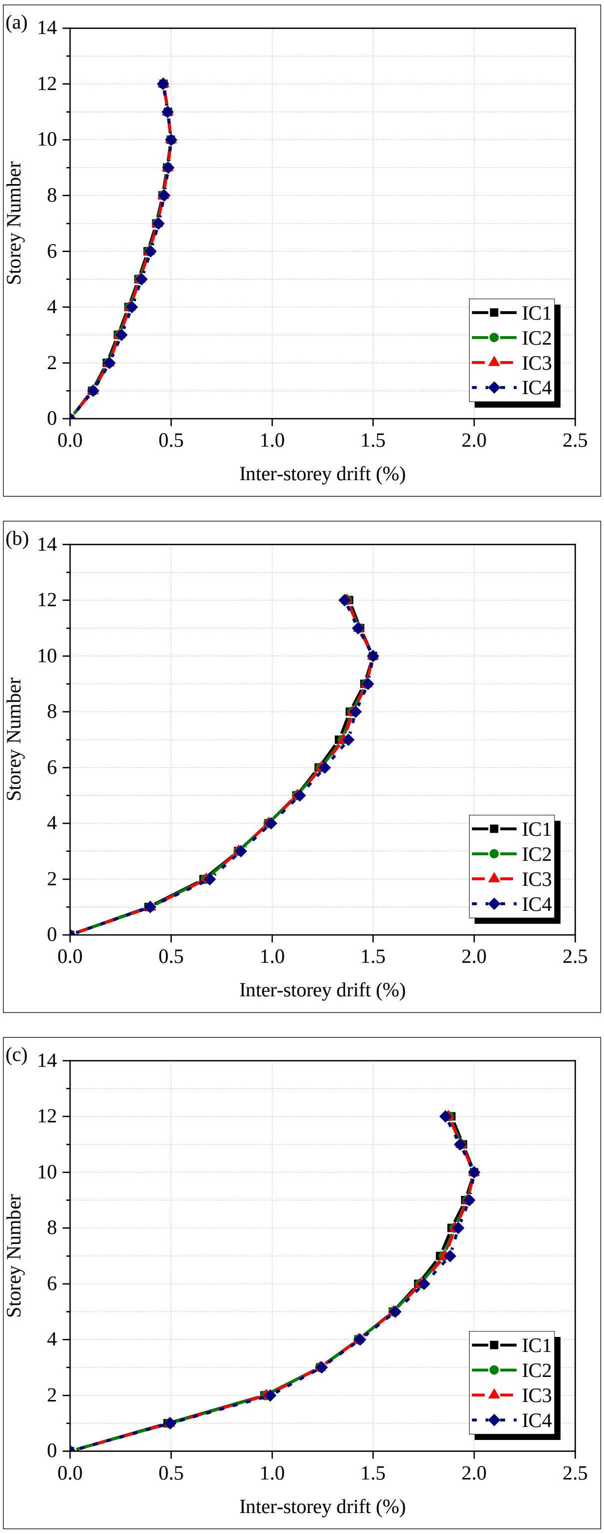

Fig. 10. Inter-storey drift ratio of the 12-storey building (a) IO (b) LS (c) CP performance levels.

That is, although changing the behaviour of inter-connection affects the inter-storey drift of the building, however, the maximum difference between the IC1 and IC4 are 6.1%, 5.2%, and 3.5%, corresponding to IO, LS, and CP performance levels. Therefore, it can be seen that the mechanical properties of considered inter-connections have no significant influence on the inter-storey drift distribution of the building.

Figure 11 (a) to (c) illustrates the influence of inter-connection's properties on the storey shear distribution of the 4-storey building corresponding to the IO, LS, and CP performance levels. The figure indicates that the stiffness of inter-connection has a negligible effect on the storey shear distribution. The figure indicates that at the IO performance level the use of IC4 inter-connection decreases the maximum storey shear by almost 10% compared to the use of IC1 inter-connection. On the other hand, compared to IC1 inter-connection, the storey shear increases by 6% and 5% at LS and CP performance levels when the IC4 inter-connection is being used in the 4-storey modular building. This illustrates that the use of the most flexible inter-connection has negligible influence on the storey shear of the considered building.

Figure 12 (a) to (c) show the storey shear of the 8-storey modular building. The results indicate that although the use of IC4 inter-connection changes the storey shear at each level, compared to the IC1 inter-connection, the maximum difference between the maximum storey shear at each level corresponding to IO, LS, and CP performance levels is 6.2%, 3.3%, and 5.1%, respectively.

This illustrates that the considered mechanical properties of inter-connections has negligible influence on the storey shear of the 8-storey building. Finally, a comparison has been made between the storey shear of 12-storey corresponding to various considered inter-connections' stiffness at IO, LS, and CP performance levels.

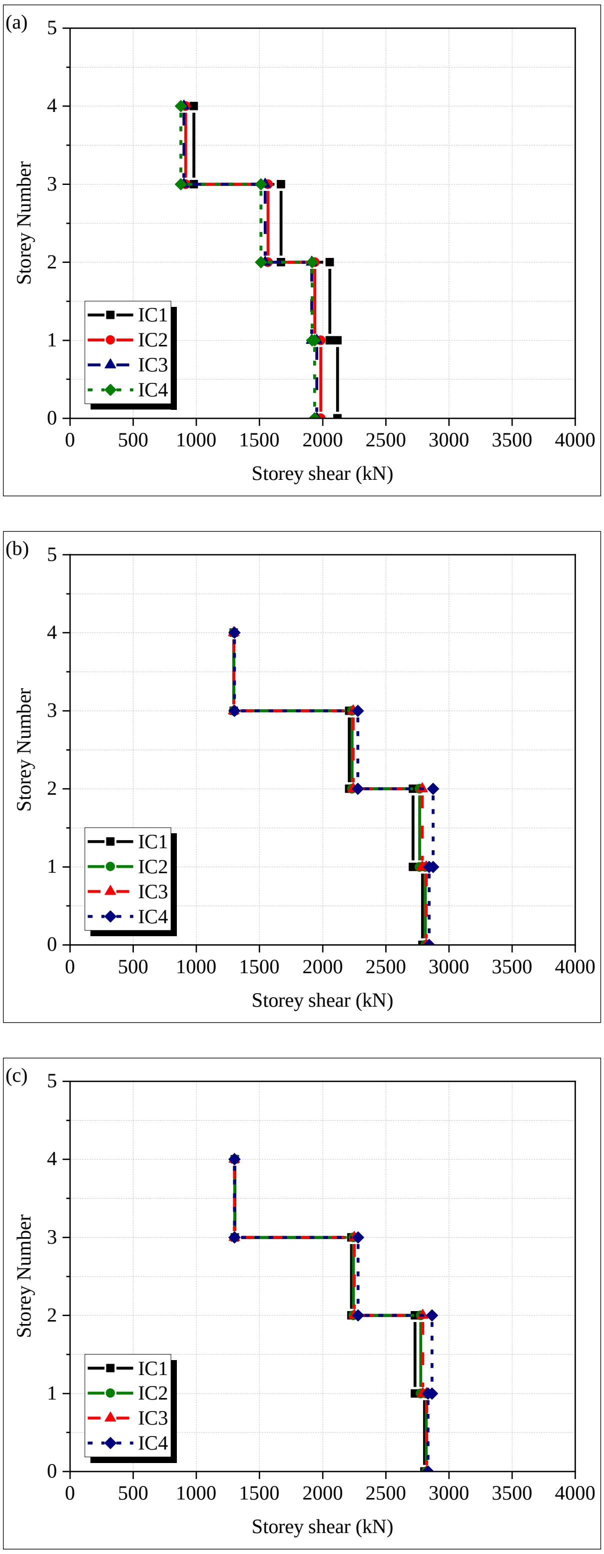

Fig. 11. Storey shear of the 4-storey building (a) IO (b) LS (c) CP performance levels.

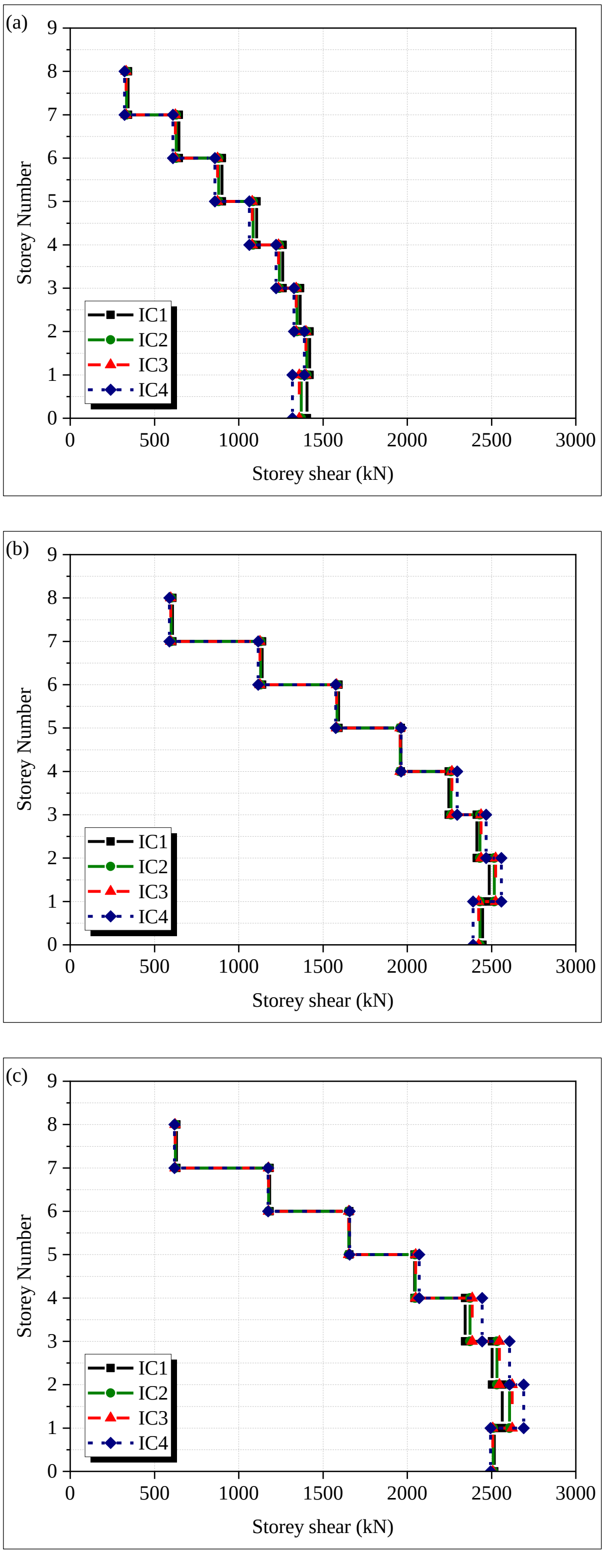

Fig. 12. Storey shear of the 8-storey building (a) IO (b) LS (c) CP performance levels.

The obtained results, shown in Figure 13 (a) to (c), illustrate that the inter-connection's behaviour has no significant influence on the shear distribution of the building, and it is the same for all inter-connections' behaviour at different performance levels.

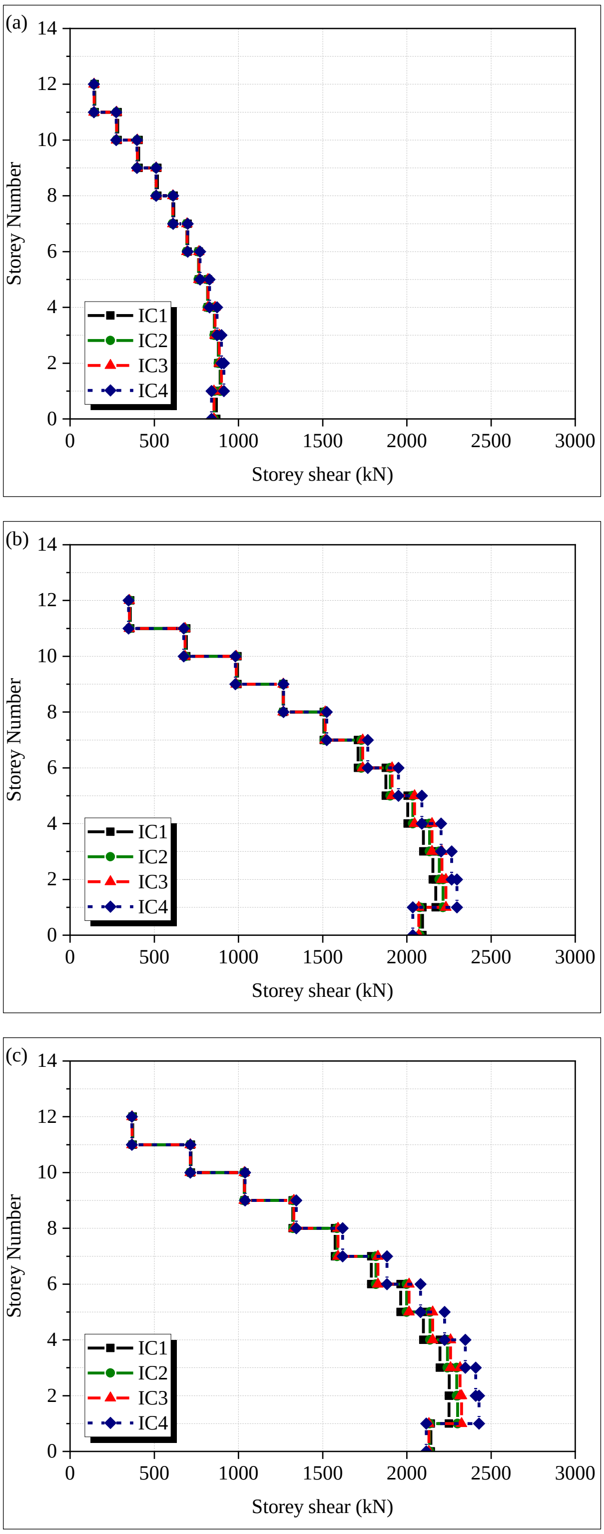

Figure 1. Storey shear of the 12-storey building (a) IO (b) LS (c) CP performance levels.

However, the use of IC4 increases the shear storey by almost 6.3%, 5.2%, and 3.3% compared to IC1 inter-connection at IO, LS, and CP performance levels. These results indicate that the mechanical properties of inter-connection have negligible influence on the lateral performance of braced frame corner-supported modular buildings. This is mainly due to the fact that the lateral stiffness of the frame is provided by braces' stiffness.

5. Conclusion

The aim of this paper is to investigate the lateral performance of braced frame corner-supported modular steel buildings (CSMSBs). The lateral bracing system is primarily provided through a combination of a dual system of the intermediate moment-resisting system as well as braced frames. The influence of inter-connections is studied for three designed corner-supported modular buildings, including four-, eight-, and twelve-storey modular buildings. Four types of inter-connections having different mechanical properties are considered for the parametric study. The lateral performance of the designed structures is scrutinised through the nonlinear static analysis. The obtained responses indicate that the increase of flexibility of inter-connections increases the natural period of the designed structures. However, the influence of flexibility is more obvious for low-rise structures. Moreover, the nonlinear static analysis indicates that the flexibility of inter-connection has no significant effect on the lateral capacity of the designed structures. The storey shear and drift of structures, obtained from nonlinear static analysis, indicate that the variation of inter-connections' properties has no significant influence on these responses. The use of IC4 inter-connections increases the drift of the 4-storey building by almost 10.1% at the CP performance level. On the other hand, this response increases by almost 3.5% for the 12-storey building at CP performance level.

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Conflicts of interest

The authors declare no conflict of interest.

Authors contribution statement

Conceptualisation, methodology, software, formal analysis, and writing of original draft preparation by Mostafa Farajian, supervision, visualization, and project administration by Mohammad Iman Khodakarami.

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