NONLINEAR BEHAVIOUR OF HYBRID MODULAR STEEL STRUCTURES WITH REINFORCED CONCRETE SHEAR WALLS

E. Bazarchi(1), A. Davaran(2), C.P. Lamarche(3), N. Roy(4), S. Parent(5), H. Fatemi(6)

(1) Ph.D. Candidate, Université de Sherbrooke, Ehsan.Bazarchi@USherbrooke.ca
(2) Research Associate, Université de Sherbrooke, Ali.Davaran@USherbrooke.ca
(3) Associate Professor, Université de Sherbrooke, Charles-Philippe.Lamarche@USherbrooke.ca
(4) Associate Professor, Université de Sherbrooke, Nathalie.Roy@USherbrooke.ca
(5) Adjunct Professor, Université de Sherbrooke, serge@steelssalg.com
(6) Ph.D., Université de Sherbrooke, Hassan.Fatemi@USherbrooke.ca

Abstract

Modular constructions are off-site prefabricated structures with transportable units that are assembled on-site. These units are linked together using mechanical connections. There are many advantages to this structural system such as the speed of fabrication, site mobilization and erection cost savings, weather independent fabrication and high factory quality control. However, the discontinuity of the diaphragm and existence of connections with slip potential and other sources of nonlinearity in a modular building system are among the subjects that lack a thorough research background. Because the exterior walls, internal partitions, floors and ceiling systems, non-structural and structural components are mostly assembled at the fabrication plant, there remains limited access to field-installed inter-modular connections, which would provide the continuity of the whole structure. In these structures, the axial and shear stiffness of inter-modular connections together with the stiffness of the floor/ceiling diaphragm of individual modules affects the response of the structure and the amount of the forces directed to the seismic force-resisting system (SFRS). In this study, the structural behavior is investigated by analyzing two types of building: 6 and 12 stories modular steel structures combined with reinforced concrete (RC) shear walls as their SFRS, through numerical modeling in OpenSees and SAP2000. The results of this study show that, for the calculation of realistic lateral deflection, the value obtained from the linear elastic analysis should be more amplified to include both the diaphragm and SFRS nonlinear responses. In the case of using HSS columns, the flexible connection assumption should also be considered.

Keywords: earthquake; steel structure; modular structure; connections; flexible diaphragm

1. Introduction

In recent years, the growing needs for precise structural quality control, life-cycle energy assessment in construction due to environmental issues, shortage and high salary of skilled labor and safety considerations, more attention is paid for prefabricated modular building units. The modular steel building concept (MSB) is well-accepted in the construction industry, in which the volumetric units, called modules, are fabricated in the shop, transported and assembled with minimum on-site activities. Modular construction is commonly used in the UK, Sweden, Japan, and the USA with a growing interest in Australia and China [1]. The volumetric term might be misleading because it may refer to temporary or demountable structures [2] for short term demands, while modular steel structures are completely different from those structures in terms of design criteria, quality and performance. The repetitive and cellular nature of modular systems makes them appealing for building structures such as hotels, schools, hospitals, and dormitories having repetitive in-plane architectural configurations. The construction time necessary to build a modular building structure is much less than in the case of a regular building. A 60% reduction in on-site construction time for modular hotels and restaurants was reported by Lawson et al. [3], emphasizing a clear economic advantage in using a modular structural system.

From a structural perspective, the two most commonly addressed topics regarding modular steel structures found in the literature are: 1) the analysis of the structural response of modular structure exposed to lateral loads such as earthquake [4], wind [5] and blast [6] loads. This topic involves the study and assessment of various lateral load resisting systems such as conventional x-braces [7], RC shear walls, steel plate shear walls [8], and double skin steel panels [9]; and 2) inter-modular connections. Several research projects found in the literature have focused on designing connections that are structurally efficient and easily connectable on-site. Despite these research efforts, currently used/proposed connections are not as efficient as expected [10].

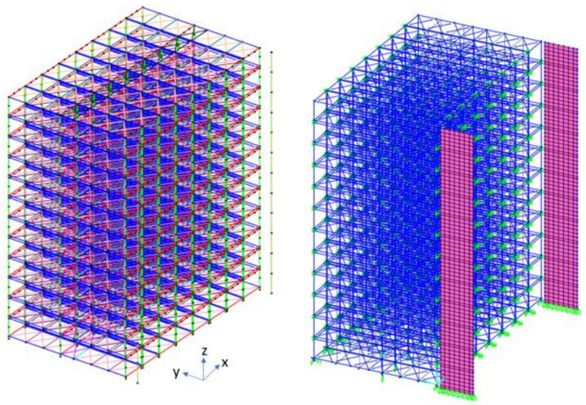
A relatively wide variety of inter-modular connection types have been proposed and examined by researchers [11-16]. For these connections, one of the main objectives was to find a configuration to ease the on-site installation effort with minimum need for access holes and openings. Inter-modular connections can be categorized into two main groups. The first group comprised of conventional, more rudimentary types of connections in which the web, flange or thin wall of structural elements of adjacent vertically aligned modules are connected directly with structural bolts [12] or using welds [11]. Also, the structural elements of horizontally aligned adjacent modules can be connected with bolts [13]. Some of these connections do not apply to interior columns in modular steel structures because of the need for more access space to perform mechanical operations [16]. The second group, comprised of more sophisticated intermodular connections, generally designed using male-female plug-in devices to connect the vertically aligned modules [15-16], or a combination of plug-in devices (for vertically aligned modules) with horizontal plates to connect the modules horizontally [14]. In the latter, some additional structural fasteners are used to tie the ceiling and floor beams to each other to avoid the extra slip or uplift. The implementation of this type of connection can be difficult, especially when the modules are transported and assembled on-site with walls and floors included. The beam to beam bolted connection, either for W or HSS sections, with or without plug-in element, are usually attractive, as both vertical (uplift issue) and horizontal connectivity are fulfilled. The presence of plug-in or male-female elements can facilitate the alignment of the piled-up modules during construction.

In this study, a relatively simple male-female connector, consistent with hollow structural sections (HSS) is used for the columns and a bolted link plate ensures the horizontal continuity between adjacent modules. The beam to beam connection configuration was not studied because one of the specific objectives of the client sponsoring the research project was to include modularity not solely in the structure, but also in non-structural components such as architectural and mechanical components. One possible solution to fulfill this objective is to limit the number of connections and access holes. For easy on-site installation, the presence of gaps between the connected parts, structural bolts, and elongated holes was almost inevitable in the proposed connection. The intra-modular connections, i.e., the connection between beams and columns in each module, are of the welded type. Though the modules are not aimed to be part of the seismic force-resisting system (SFRS), bending moments develop in the connections under lateral deflections in the building structures, which might have an adverse effect on the steel frame members. The SFRS in the studied structural system is comprised of reinforced concrete (RC) shear walls. The in-plane shear forces that develop due to seismic and wind actions are transferred to the SFRS through in-plane diaphragms/floor systems that are included at the bottom of each module in combination with an in-plane truss located in the floors of the central corridors located at each story level. The inter-modular in-plane shear transfer between the modules is resisted by a special connection that is not presented herein.

The main purpose of this paper is to investigate the displacements of a six and twelve-story modular building structures under lateral seismic loads, using the SAP2000 and OpenSees finite element (FE) softwares. Nonlinear static pushover analyses were performed to investigate the nonlinear behavior of the building structures. A wide variety of local nonlinear behaviors such as slip-contact in the inter-modular connections, local deformations in the HSS columns at the intra-modular connections, elastoplastic behavior of the steel and concrete materials used to model the beams, columns, and shear walls are included in the models.

2 Finite element models of the building structures

A six and twelve-story modular steel-framed structures including reinforced concrete shear walls as part of their lateral resisting systems are studied. The twelve-story building structure modeled in SAP2000 is presented in Fig. 1. The schematic 3D view of a single structural module designed for gravity loads is shown in Fig. 2.



(a) (b)

Fig. – 1 FE models of the twelve-story modular building: (a) OpenSees (with nonlinear fiber element shear walls); (b) SAP2000 (with nonlinear shell element shear walls)

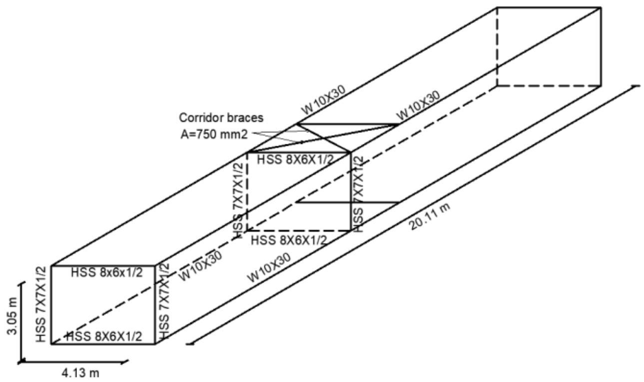


Fig. – 2 Schematic view of a single module

The concrete shear walls and steel frames are designed according to CSA-A23.3.14 [17] and CSA-S16-14 [18], respectively. For the seismic loading of the structures, NBCC2010 [19] is used and the structures are assumed to be located on soil type B in Gatineau (Qc), Canada. The seismic design parameters for the proposed site are provided in Table 1.

Table 1– Seismic design parameters from NBCC2010.

PGA	S_a (0.2)	S_a (0.5)	Sa (1)	S_a (2)	R_o	R_d
0.32	0.63	0.31	0.14	0.046	1.3	1.5

A typical plan view of the models with a section cut showing the load components is shown in Fig. 3.

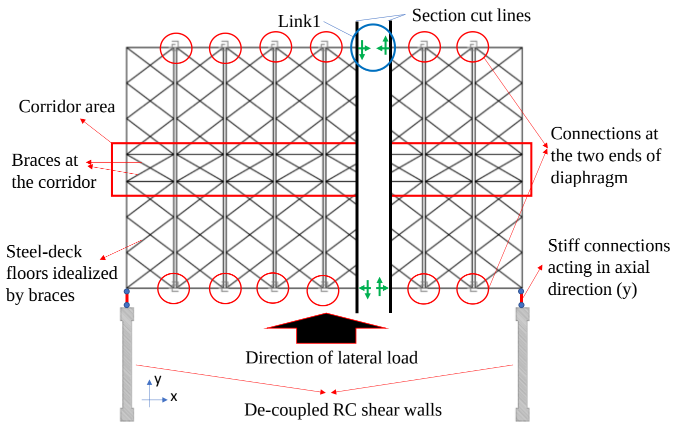


Fig. – 3 Typical plan view of a structural FE models

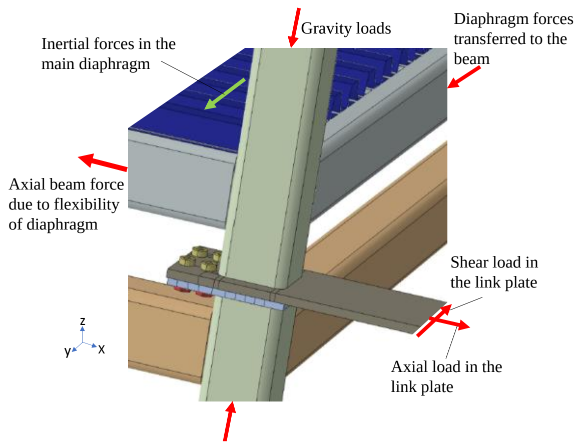


Fig. – 4 Seismic load transfer (Link 1)

In Fig. 4, the free body diagram of Link1, also presented in Fig. 3, connecting two modules stacked vertically, to two adjacent modules, is shown when the structure is submitted to a lateral seismic load in the y-direction. In Fig. 4 the inertial forces developing in floor diaphragms are transferred to the main floor beams in the x and y directions and axial and shear force components transit through the link plate. Parasite bending moments, shear and axial force components are not represented in Fig. 4 for clarity.

The in-plane bracing elements in the corridor area (middle bay) are used to increase the in-plane stiffness of the floor diaphragm, therefore, mitigating the shear demands in the inter-modular connections and the steel deck panels. The floor of each module is made from Canam P-3606, 24 gauge (6.4 mm), bare steel deck panels. In the FE models, the floor panels are idealized using equivalent truss elements for the sake of simplicity and to reduce computational cost. The beam to HSS column connections is of the welded type in both principal directions. The continuity between two adjacent modules is ensured by inter-modular connections located at each corner of the modules. In the FE models, the shear-walls are decoupled and connected to the frame nodes using geometrical constraints defined in the horizontal direction. The seismic parameters of the 6 and 12-story building structures and the design base shear for two soil types are presented in Table 2.

Table – 2: Properties of the FE structural models

Model	Fundamental period y-dir. (seconds)	Seismic weight (kN)	Design base shear (B soil) (kN)	Design base shear (C soil) (kN)	Inter-story height (h_i) (m)	Structure's height (H) (m)
6-storey	0.70	12914	1472	2214	3.05	41.0
12-storey	1.24	26477	1804	2806	3.05	20.5

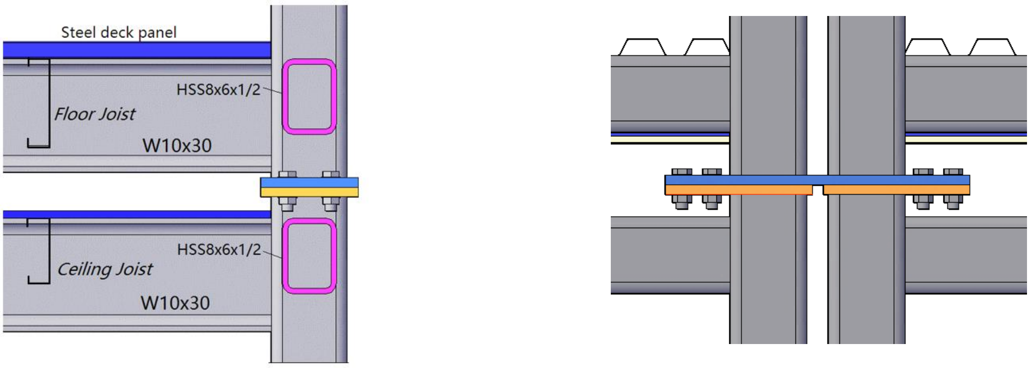
3 Structural behavior of the inter- and intra-modular connections

In this section, the nonlinear behavior of inter-modular connections evaluated by FE analyses is presented and explained. The structural behavior of the intra-modular connections is also discussed. Finally, the analytical model employed for RC shear wall and the method of validation are presented.

3.1 Inter-modular connections

In a particular module, to avoid the extra costs of building two (bracing systems and/or in-plane diaphragms) at each story level), one must choose between including the in-plane load resisting system at the floor level or the ceiling level. When all adjacent modules stacked horizontally at a given story level are linked together, the ensemble of the individual in-plane load resisting systems forms a deep in-plane diaphragm that is used to transfer the lateral loads to the foundation through SFRS. In this study, the in-plane diaphragms are located at the floor levels. When inertial loads develop at each floor level during a seismic event, most of the horizontal loads are carried to the inter-modular connections according the free body diagram presented in Fig. 4. Nevertheless, a certain amount of the lateral loads inevitably transits to the inter-modular connections through the ceiling beams, especially under wind-induced lateral loads. This particular load transferring mechanism is also discussed in this paper.

The horizontal link connection (encircled in Fig. 3) is comprised of a plate, which connects the column cap plates via 25.4 mm diameter A325 fasteners. The vertical alignment of the piled-up modules during the construction is ensured by a male-female connecting mechanism positioned at the end of columns. A tapered stocky projecting element, which is bolted to the cap plate of the corner columns using four 22mm tightened bolts resulting in a slip-critical connection acts as the male component (Fig. 5).



(a) (b)

Fig – 5 Intermodular connection: (a) side view; (b) front view

This male component of the connection that is embedded in the HSS column located at the upper story acts as a shear transfer mechanism, which transfers a portion of the shear force coming from the upper story columns to the story level below. The details concerning this part of the connection could not be included in this paper due to a confidentiality agreement signed with the industrial partners involved in the research project. Because the inclusion of small gaps is always necessary to compensate for the fabrication tolerances, there is a possibility that upper column may slide on the link plates, until the connection is engaged through the contact between the projecting element and inner face of the HSS column.

3.2 Structural behavior of inter-modular connections

The modeling of the structural behavior of the proposed inter-modular connection is idealized by uncoupling two distinct functionalities. The first functionality corresponds to linking two side by side modules horizontally. The second functionality to linking two modules stacked vertically. A schematic view of the idealized structural modeling of an inter-modular connection is presented in Fig. 6.

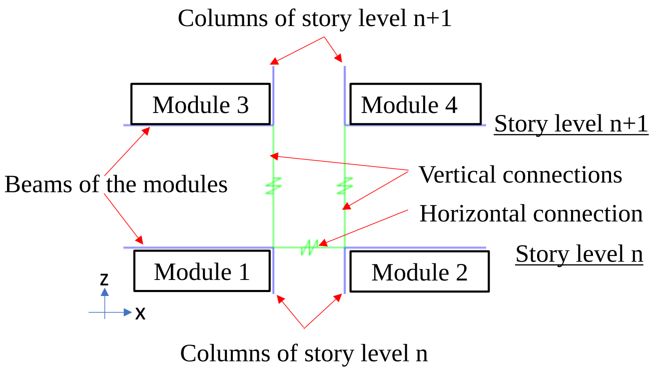
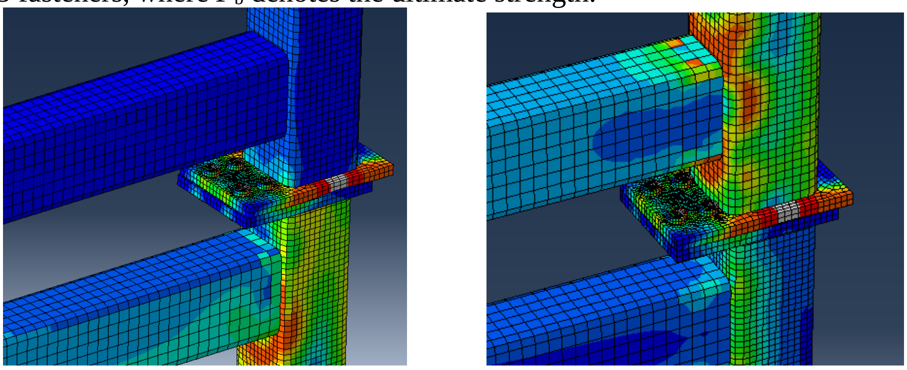


Fig. – 6 Modeling of an inter-modular connection between four adjacent modules

The FE model of a typical connection modeled using the software ABAQUS is presented in Fig. 7. Solid 8-node elements (C3D8R) are used for the plates and bolts. For the bottom HSS column, 4-node shell elements (S4R) are used. For the upper column, where sliding can potentially occur, solid 8-node elements are used because a more accurate sliding response was obtained using this particular type of element. The thickness of plates t_p, the yield stress F_y of the steel material and the steel-steel frictional coefficient μ are assumed to be 22.2 mm, 350 MPa and 0.3, respectively. A pre-tension load of 0.8 x 0.75 x F_b is applied on the ASTM-A325 fasteners, where F_b denotes the ultimate strength.



(a) (b)

Fig. – 7 Connection model: (a) Load transferred through ceiling beam; (b) Load transferred through floor beam

Friction, sliding and contact behaviors have been included in the FE model to make the model as accurate as possible. In the analyses, a gravity load of 25 kN is applied to the upper column. This load corresponds to the tributary weight carried by the connection assuming there is only one story of modules stacked above. The in-plane force-deformation response obtained by FE analysis for the load transferred via the floor beam is shown in Fig. 8. This curve is used to define the behavior of the vertical link elements in the FE models of the building structures. As depicted graphically on the response curve, the bottom of the upper column slides by 4 mm over the link plate before the male-female mechanism is engaged. The axial and shear behavior of the link plate (corresponding to the horizontal link elements in the structural model) is illustrated in Figs 9 and 10, respectively.

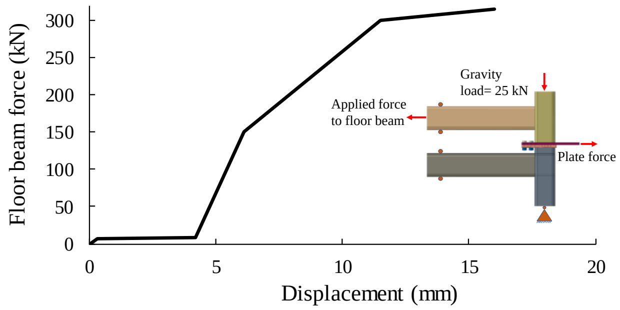


Fig. – 8 Shear behavior of the vertical connection

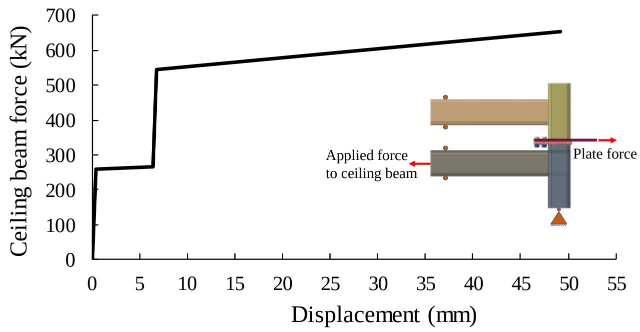


Fig. – 9 Axial behavior of the horizontal connection

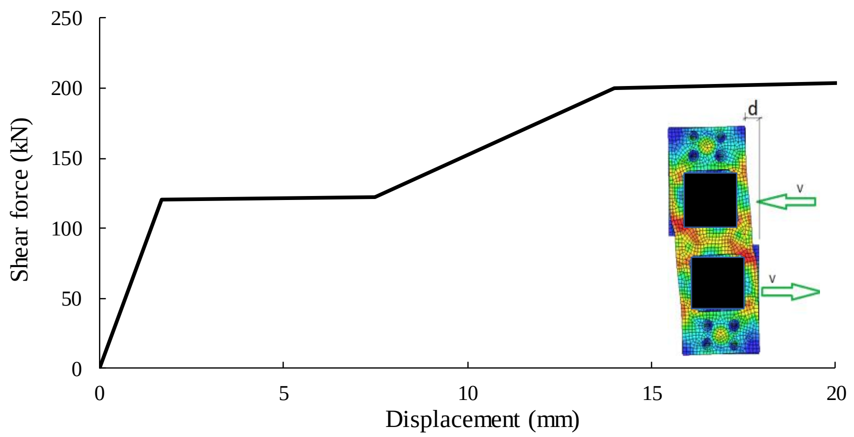


Fig. – 10 Shear behavior of the horizontal connection

In Fig. 9, the axial force in the horizontal link plate linearly increases up to 260 kN until slipping between the link plate, and the cap plate occurs, and the force remains constant (dry friction). Then, when the gap between the bolt holes and the plates is closed, the force increases again until the link plate yields in tension at a force of about 530 kN. Fig. 10 shows that for the horizontal link, slip occurs at a shear load of about 120 kN. After the closing of the gaps and full engagement of the connection, the shear force increases again and the plate yields. At an effective strain level of 0.2 for the plate, the connection carries a shear force of about 238 kN, which is assumed to be the ultimate shear strength level in this study.

3.3 Intra-modular connections

Welded beam to HSS column connections (intra-modular) cannot be considered as perfectly rigid connections because of the presence of local deformations at the connection interface under loading. Therefore, extra zero-length elements are used in the structural models to account for the added flexibility in the intra-modular connections. To this end, a single frame of a module was modeled in ABAQUS, OpenSees and SAP2000. The zero-length element properties in OpenSees are determined by matching the push over response curves resulted from the ABAQUS detailed volumic FE analysis and fiber section beam-column model. Using a similar procedure in SAP2000, modified plastic hinge properties were adopted for the beams. The pushover curves of a single frame model, before and after calibration with detailed FE analysis, are depicted in Fig. 11.

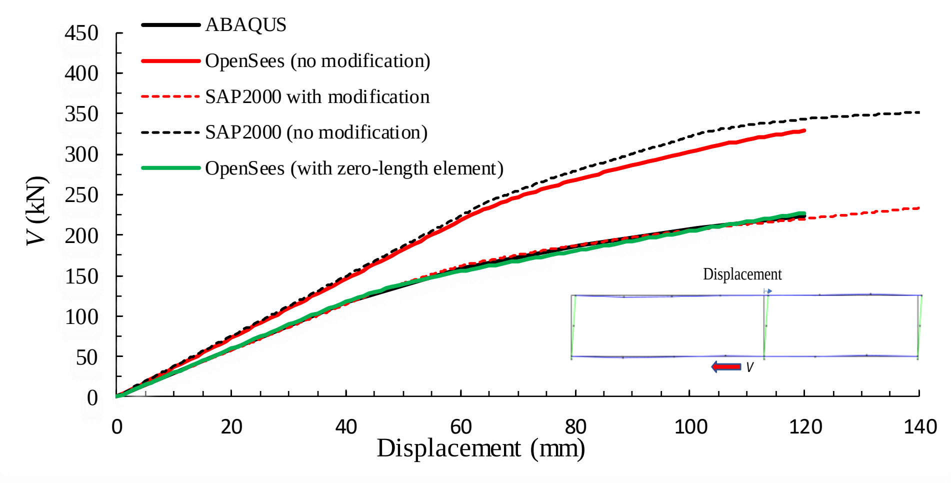


Fig. – 11 Calibration results for a single 2D frame with plastic hinges and beam-column with flexible connections

4 Validation of the numerical model used for the RC shear-wall

RC shear walls were chosen as SFRS for the studied modular structures. Having a precise and efficient method for modeling the shear wall is a valuable asset to decrease computational cost. Different modeling techniques for RC shear walls, such as nonlinear layered shell element (SAP2000), nonlinear FE (VecTor2), fiber section and distributed plasticity (OpenSees), and fiber section plastic hinge (SAP2000) are examined in this study. Shear wall sections designed for the 12 and 6-story modular buildings are shown in Fig. 12.

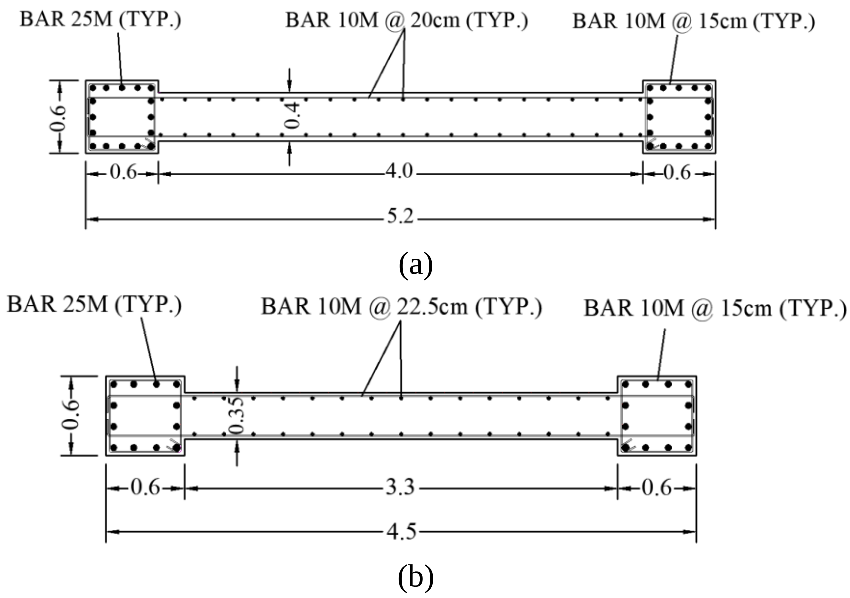


Fig. – 12 The details of designed shear-walls for modular buildings: (a) 12- storey model; (b) 6-story model

An isolated shear wall of the 12-story 41m height modular building was modeled with four different approaches: (1) nonlinear smeared crack model in VecTor2, (2) nonlinear fiber element in OpenSees, (3) SAP2000 with multilayer nonlinear shell element and (4) SAP2000 with fiber section plastic hinges defined at each story level. The VecTor2 model, being the most accurate of the four numerical models [20], is considered as the reference model. Two types of materials are used for defining the behavior of concrete material: confined concrete for the boundary columns and un-confined concrete for the wall. The nonlinear behavior of confined and unconfined concrete material used in the models is presented in Fig. 13. Also, the nonlinear material behavior for reinforcement steel is presented in Fig. 14.

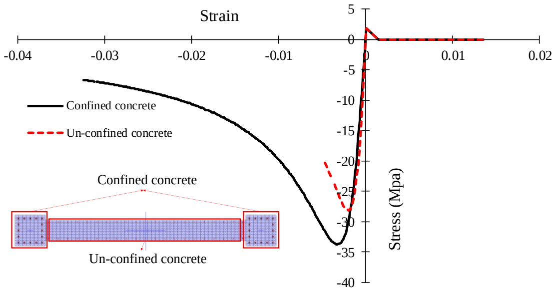


Fig. – 13 NL behavior of confined and unconfined concrete used in the models

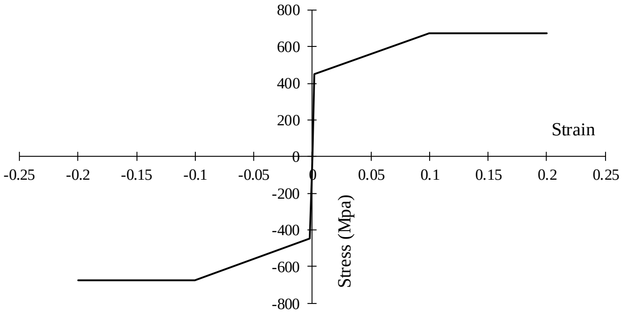


Fig. – 14 NL behavior of steel rebar in RC shear walls

The pushover curves of the shear wall under a triangular lateral load pattern (in the absence of gravity loads) are shown in Fig 15.

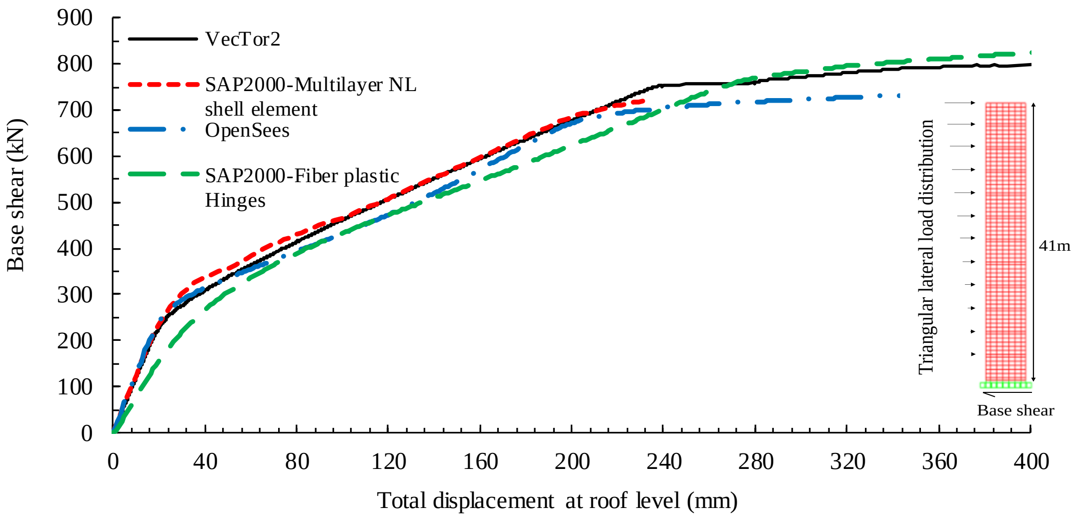


Fig. – 15 Push curves of an isolated 12-story shear wall

According to Fig 15., the best match exists between the multi-layer nonlinear shell element and VecTor2 push over curves. However, the nonlinear multi-layer shell element is prone to divergence problems and, therefore, the analysis could not converge past a roof displacement of 240 mm. The OpenSees fiber and SAP2000 plastic hinge models predict the push over response with a reasonable computing time and accuracy. Therefore, these modeling methods were adopted in this study.

5 Response of the 12-story building structure model

The results from nonlinear static analysis of a 12-story modular building obtained from SAP2000 and OpenSees software are summarized in Fig. 16. The modeling methods are described earlier to emulate the connections and shear wall behavior. The pushover curves presented in Fig. 15, obtained from the SAP2000 and OpenSees models, closely match each other. The model based on fiber plastic hinges and fiber section distributed plasticity have advantages compared to the other method due to less computational cost involved in the analysis of shear walls.

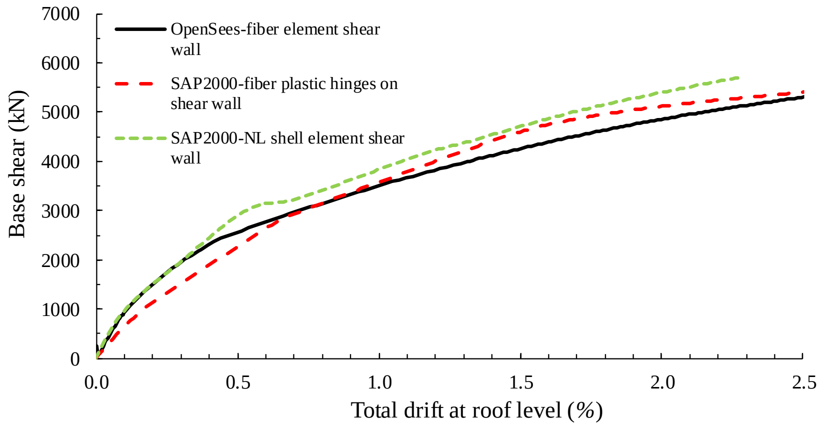


Fig. – 16 Validation of the pushover curves for the 12-storey fully nonlinear models with intra-modular flexible connection

In Fig. 16, after the concrete of the RC shear wall loses its tension capacity at about 0.1% drift, a decrease in stiffness can be observed. As the pushover analysis progresses, the shear wall reaches its moment resisting capacity at its base when a 0.6% drift is reached. After the capacity of the shear wall is reached, the overstrength in the structure is due to the elastic stiffness of the modular framing elements. It should be noted that before a 0.9% drift is reached, no plastic excursions are observed in the modular framing elements.

6 Effects of level of sophistication of the FE models on lateral response

The response of modular steel structures including the shear walls depends on the structural behavior of its components, i.e., the stiffness and resistance of the steel members, the RC shear walls and the connections. The prediction accuracy of any structural model relies on the level of numerical sophistication used in defining each component. As a design engineer, it is worthy of attention to know the significance and the impact of each component's characteristics on the overall response of the building. Furthermore, the structural integrity and deformation of the diaphragm in modular steel structure are of great importance and are dependent on the behavior of the inter and intra-modular connections. If premature deformation, like a large slip, occurs at the loads lower than the yielding load of SFRS, this normally cannot be captured through the traditional linear elastic analysis and might lead to an error in the component design. To examine the effects of each item on the lateral response of the 6- and 12-story structures, five sub-cases are considered as shown in Table 2. Nonlinearity sources are added one at a time, in each case. The displacement for each case is compared to the displacement of the basic linear model in which the shear wall second moment of inertia is taken as $0.35I_g$ and $0.7\ I_g$, corresponding to cracked and un-cracked states (case 1), where I_g is the second moment of inertia of the uncracked section.

Table 2 Models used to investigate the effects of structural nonlinearities and flexibility of the connections

Case No.	Walls	Frames	Inter-modular connections	Intra-modular flexible connections
Case 1	Linear	Linear	Linear	No
Case 2	Nonlinear	Linear	Linear	No
Case 3	Nonlinear	Nonlinear	Linear	No
Case 4	Nonlinear	Nonlinear	Nonlinear	No
Case 5	Nonlinear	Nonlinear	Nonlinear	Yes

The pushover response for all cases of the 6- and 12-story models are illustrated in Figs 17 and 18. The structures were designed for a type B soil. The design base shear level for type B and type C soils are also presented in the figures. The base shear *vs* drift ratio, corresponding to the ratio between the roof displacement (D_{roof}) and the height of the structure (H) are summarized in Table 3.

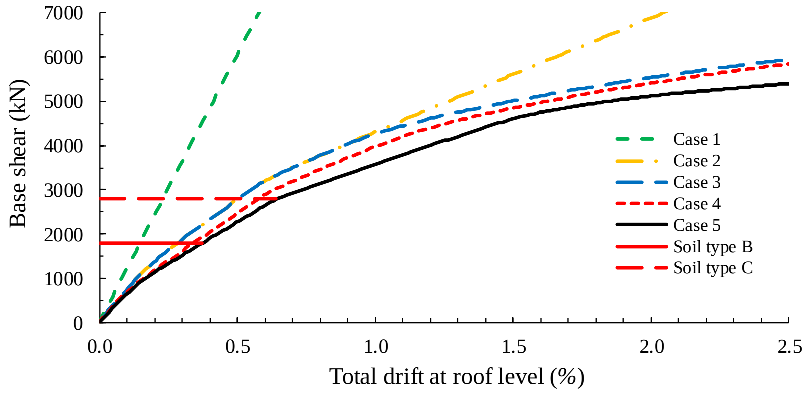


Fig. – 17 Push curves for different cases of 12-storey model

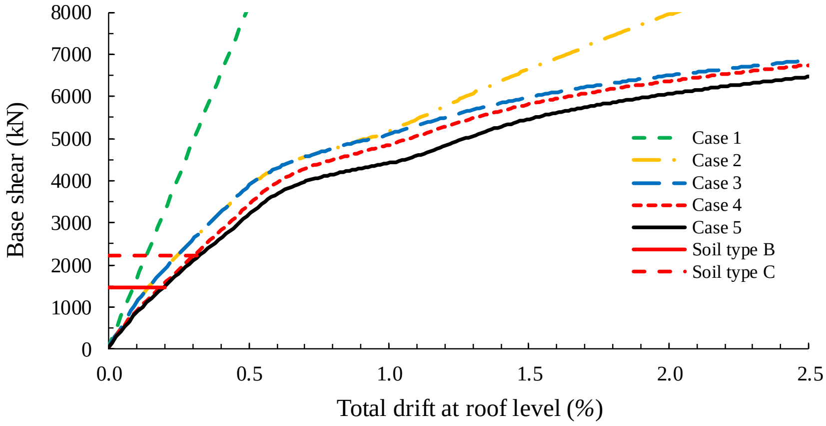


Fig. – 18 Push curves for different cases of 6-story model

Table 3 Summary of overall structural drifts for different cases

Model type	Soil type	Case 1 drift (%)	Case 2 drift (%)	Case 3 drift (%)	Case 4 drift (%)	Case 5 drift (%)
6-storey	B	0.147	0.282	0.282	0.343	0.375
	C	0.229	0.505	0.505	0.578	0.648
12-storey	B	0.084	0.137	0.137	0.178	0.188
	C	0.133	0.242	0.242	0.302	0.323

The ratio of inelastic to linear elastic displacements of all the nonlinear simulations of the 12- and 6-story models are presented in Figs. 19 and 20, respectively. The horizontal red line in the figure is an indicator of the code amplification factor, i.e., R_oR_d (= 1.5 × 1.3 = 1.95). According to these figures, for the 12-story and 6-story models located on a type B soil (for which the building structures were designed), the amplified elastic displacement obtained is conservative for the cases when only the nonlinearity of RC walls and frames are considered. On the other hand, when the nonlinearity of the inter-modular connections is considered in the models, the nonlinear displacements exceed the code recommended [19] amplified elastic displacement. In an attempt to quantify this amplification, the realistic maximum displacement can be expressed using Eq. (1):

$$D_{Inelastic}=\alpha.\beta.R_o.R_d.D_{Elastic} \quad (1)$$

Where, in modular steel structures, α and β are the displacement amplification factors related to the nonlinearity of inter-modular connections and flexibility of intra-modular connections.

According to Figs. 19 and 20, for the 12-story structure of this study, the suggested values for α and β are 1.21 and 1.09, and for the 6-story structure are 1.3 and 1.06, respectively.

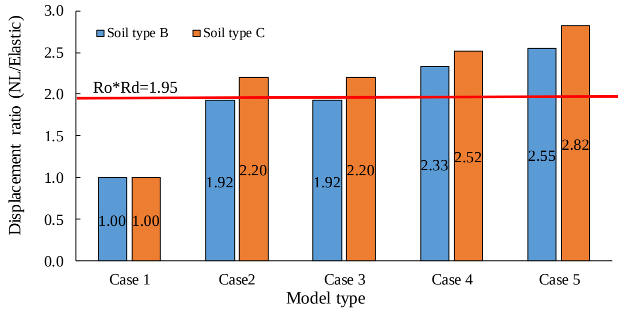


Fig. – 19 Ratio between the lateral deflection obtained from the non-linear and linear elastic 12-story models.

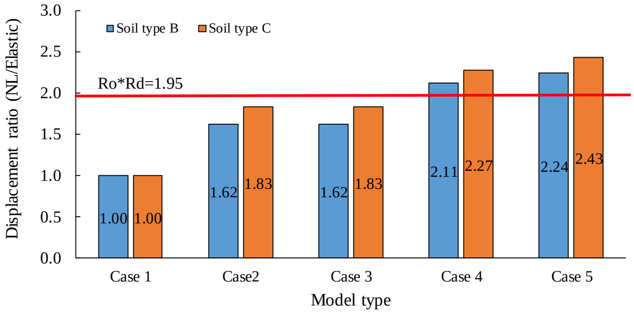


Fig. – 20 Ratio between the lateral deflection obtained from the non-linear and linear elastic 6-story models.

The proportion of the displacement related to each modeling assumption expressed as a percentage of the total drift of structures are presented in Fig. 21.

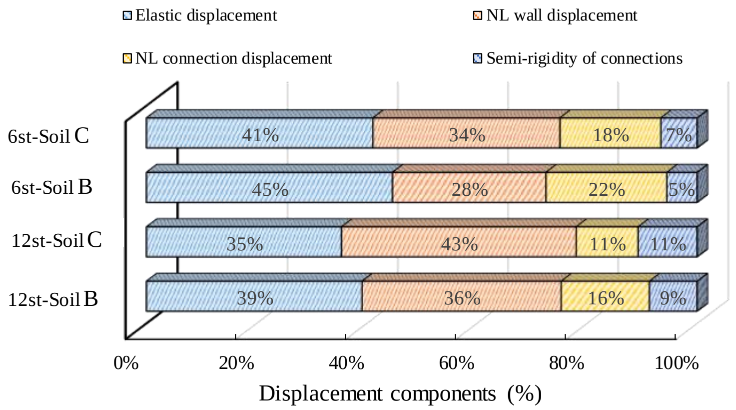


Fig. – 21 Proportion of the displacement related to each modeling assumption

Regarding this figure, no plasticization happens in the steel frame for both 6- and 12-story models even at the design base shear of soil type C. Fig. 21 shows that in the case of the 6-story structure, the displacement response is more influenced by the inter-modular connection nonlinearities, compared to the 12-story structure. From Fig. 21, it becomes evident that the flexibility of the beam to column (intra-modular) connection has a bigger impact on the roof displacement in the case of taller building structure.

7 Conclusions

Nonlinear static analyses were performed on container-type steel modular structural systems using the SAP2000 and OpenSees softwares. Different sources of inelastic behavior such as material nonlinearity, inter-modular connections including slip and flexibility in the intra-modular connections were included in the analyses. The results obtained from the nonlinear analysis of 6- and 12-story modular building structures including those nonlinear behaviors are as follow:

- The maximum roof displacement of 6- and 12-story buildings at the design base shear for soil type B increased by 30% and 21%, respectively, as influenced by the slip behavior of the inter-modular connections. The gap in the connection which inherently exists for the easy on-site assemblage activity, mobilizes the slip behavior to happen at an earlier stage than yielding of the shear walls.

The codified amplification factor of $R_o.R_d$ was not found to cover the mentioned extra inelastic deflections resulted from connection behavior.

- The local deformation of the HSS column face attached to a W or HSS beam produces a flexible connection condition and increases the lateral deflection by 6% and 9%, respectively, for the 6- and 12-story modular buildings.
- Using plastic hinge with sectional fiber discretization for the RC shear walls can lead to computationally efficient analytical models with acceptable accuracy, in comparison with the other refined modeling technique such as layered nonlinear shell elements.

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