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Review

Numerical models for lateral behaviour analysis of cold-formed steel framed walls: State of the art, evaluation and challenges

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ABSTRACT

Cold-formed steel (CFS) shear walls are the primary lateral load resisting components in lightweight steel framed (LSF) structures. The development of increasing complex LSF structures, laterally supported by CFS walls, demands sophisticated modelling techniques for design and optimization that typically involves inherent material and geometric nonlinearities caused by large deformations. Determining the performance of CFS shear walls and accurately establishing their behavioural model are the foundations of obtaining effective responses of CFS structure under extreme loading conditions. Great progress has been made on the theories and applications of the numerical models for analysis of the lateral behaviour of CFS wall systems during the past several decades, and quite a great number of numerical models have been developed for simulating the behaviour of CFS shear walls in the literature. This study provides a comprehensive review on the numerical developments made in this area as published in leading journals, high impact conferences and codes' provisions in the area, and looks at the challenges and gaps that need to be addressed in future research studies. The numerical models for analysing the lateral behaviour of CFS shear walls including their strengths, weaknesses, limitations employed behavioural models, contributing factors, and parameters and functions influencing their performance are discussed and compared with each other. The existing models are grouped into two categories: micro modelling methods, which simulate fine-scale details; and macro modelling methods, which amalgamate details into selected categories for further simplification.

1. Introduction

Cold-formed steel (CFS) structures have been used extensively in construction industry due to their exclusive advantages such as being cost-effective, lightweight, easy to install, low maintenance and recyclable materials. As a result of high structural, technological and environmental performance of lightweight steel framed (LSF) constructions, the use of CFS shear walls in high seismic areas is rapidly growing compared to their other counterparts [1]. The increasing need for mid-rise CFS frames in recent years has also led to numerous research activities in order to improve the behaviour of these systems in compliance with the regulations [2].

Many studies have investigated the problem of lateral or seismic response of CFS structures by defining the performance of wall panels. In the literature, CFS walls are classified into shear walls (sheathing-braced walls) and strap-braced walls, which are mainly covered with sheets of steel, wood, gypsum, cement board, etc, or braced with other components, to provide a seismic resisting mechanism in LSF construction. Both systems can be used in the low to moderate seismicity zones, when they are designed according to relevant standards (FEMA-450 [3], TI 809-07 [4], ASCE 7 [5], AISI [6–14], AS/NZS 4600 [15] and IBC [16]). Section 14.1 in ASCE7 [5] indicates that the response modification factor for strap-braced walls is much lower than sheathing braced systems. In the past decades, the lateral behaviour of LSF systems made great improvement with the fast development of the discretization techniques such as Finite Element (FE) method. Yet CFS framed shear wall failure mechanisms still cannot be accurately predicted, and the accurate computational modelling of CFS framed shear walls requires further research.

In the last decade, a large amount of experimental studies [17–27] have been conducted in relation with the lateral behaviour of CFS structures and determination of lateral strength and ductility of these systems. While experimental testing is the most reliable yet expensive method for evaluation of the lateral behaviour of CFS framed structures, increase in computational power along with the growing demand for accurate modelling and realistic simulations of complex LSF structures requires more comprehensive models for predicting the behaviour of these systems. In such realistic simulations the errors of the virtual

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models should be small, and different aspects of the structure must be taken into account.

In recent years, development of numerical models for the study of CFS framed shear wall structures is being dramatically increased, which indicates a growing interest towards research in this area. In fact, a fairly great number of numerical models have been developed for simulating the behaviour of CFS shear walls in the literature; each possesses its own strengths, weaknesses and limitations. This paper reviews the numerical methods used for modelling the lateral performance of CFS framed wall structures in the literature, and discusses their positive and negative aspects, limitations, their applicable software, and challenges for simulation of different scenarios. To that end, the existing models are classified into macro modelling and micro modelling methods, and each is discussed within their own context. Then a comparative discussion on both macro and micro categories is carried out in order to evaluate their effectiveness, positive and negative aspects, and their accuracy. The study only focuses on numerical models for CFS framed shear wall structures acting as lateral resistance systems for buildings. Therefore, pure theoretical and mathematical studies as well as studies on individual, independent and stand-alone CFS members are not within the scope of this study.

2. Classification of numerical methods

Many structures are too complex to be analysed by analytical or classical techniques and, therefore, numerical analysis is generally utilized. The FE method is the most widely used numerical technique for structural analyses [28–32]. For the analysis and design of complex CFS structures with relatively large deformations and instability issues, selecting an appropriate computational technique plays a substantial role. In the structural analysis, the finer FE methods provide more accurate results, with higher computational effort costs though. When dealing with large structural dynamic problems such as studying the effects of earthquake loads on tall buildings, a large amount of simulation is required during the design and analysis stage, which results in extremely large number of describing equations and consequently very slow convergence. An effective approach for reducing the computational complexity of these models in numerical simulations is Model Order Reduction (MOR) techniques. MOR techniques lower the computational complexity of large-scale and/or dynamical systems, by a reduction of the model's associated state space dimension or degrees of freedom, through computing an approximation to the original model. MOR techniques are useful for studying large-scale complex systems, whose behaviour can be described by interactions of a number of interconnected subsystems. In structural analysis, the topology of the reduction would have some sparse structure to preserve the original structures topology through appropriate clustering method. Fig. 1 shows how a large structure's topology model, made of a large number of nodes and members, can be simplified trough model reduction and clusterisation [33].

The existing MOR methods are classified into some categories. If the reduced model is obtained by removing parts of the physical coordinates of the full model, the MOR technique is called physical coordinate model reduction, which is the most straightforward method and commonly used in structural analysis. All other non-physical coordinates such as modal coordinate and the Ritz coordinate are generally referred to as generalized coordinates. In the structural dynamics community, MOD techniques have been widely employed in complex global-local analysis, optimization and structural vibration and buckling [34].

Although FE methods themselves could be assumed as a class of MOR techniques [34], in the literature MOR methods are mostly referred to techniques replacing the large scale original model by a significantly smaller one, while maintain characteristic properties of the former and approximate its transfer behaviour as much accurate as possible. MOR techniques are mostly employed in conjunction with FE methods to facilitate solving complex problems. In fact, in computational mechanics, the numerical modelling of structures can be classified as two major groups: (1) the models simulating fine-scale details, known as micro models; and (2) those amalgamating details into select categories being used to quickly capture the essential features of a structure; known as macro models. These two strategies refer to different fields of applications: micro models are applicable when the scope of the study is the local behaviour of the structures and elements, while macro models are used when a global behaviour of the structure is required. Micro and macro models can be used together to study different aspects of a problem [35].

Due to the thin-walled nature of the structural elements used for building CFS structures, the accurate analysis of such systems is mainly performed using detailed micro modelling. Yet, micro modelling of buildings made of a relatively large number of CFS elements takes a lot of time and effort. Therefore, the development of a strong macro analysis method for CFS structures has widely attracted attentions in the past few years.

In the study of CFS systems, micro modelling methods, also called as detailed models, are those modelling the structures while considering all the components and interactions, including CFS framing members, sheathings, the connection between the framing members and the sheathing, as well as attachments. In this approach, the nonlinear behaviour of structure is usually interrelated with nonlinear behaviour of the boundary conditions, elements and connections; therefore, an appropriate basic behavioural model of the elements, usually obtained from experimental data, is required. The accuracy of micro modelling primarily depends on the type, size, and number of elements used to model a CFS structure. Micro modelling approach is usually used for smaller CFS structural elements, with strongly heterogeneous states of stress and strain. This approach provides possibility of real simulation of the CFS frames, with local effects in each material and element as well as at contact.

As CFS structure becomes more complex with larger number of elements, the required order of the model becomes higher that dramatically influences the computational efficiency of analyses. In those cases, reducing the order of model is a useful and practical solution. In macro modelling, also known as simplified methods, the complex building components are simulated as equivalent structural elements, in which the adopted properties of equivalent element are corresponding representatives of the real structure. Macro models remarkably simplify the analysis of complex structures; thus, can be effectively used for dynamic responses of larger CFS structures and those with sub-system consisting of wall panels and their connection to other adjacent subsystems, such as panelised buildings [1,36,37]. Macro models are particularly appropriate when the structure is composed of elements with adequately large dimensions, so that the stresses across or along a macro element is essentially uniform, negligible and/or of not much

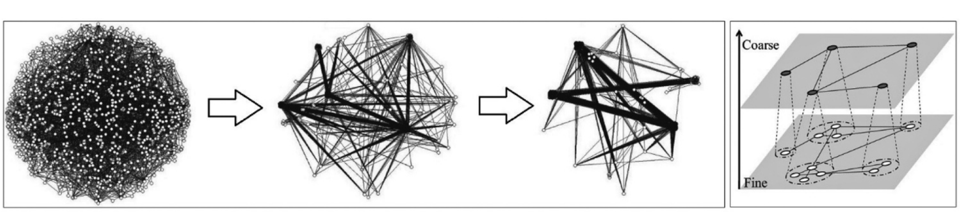


Fig. 1. Model order reduction through structure's topology clusterisation.

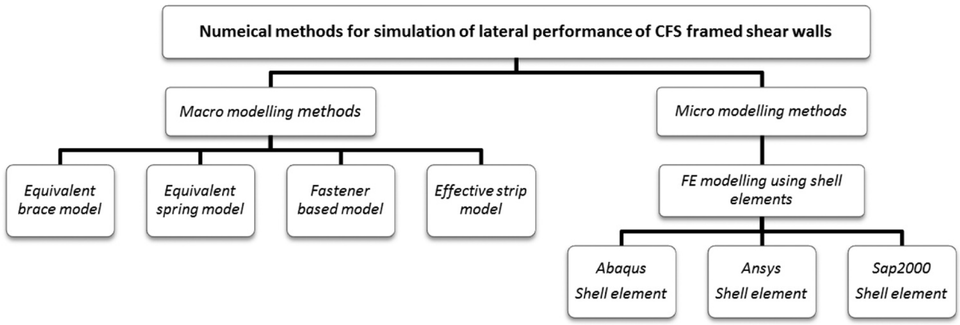


Fig. 2. Classification of numerical methods for CFS framed shear walls.

importance. Macro models are most applicable when a certain level of both accuracy and efficiency is needed.

A reasonably great number of macro and micro models have been developed for simulation of CFS framed wall structures under lateral loading. These models offer different levels of complexity, precision, efficiency, strength and applicability. A comprehensive database comparing different modelling approaches seems to be essential for the future development of more effective and comprehensive modelling methods for CFS wall structures under lateral loads. This study will first classify the existing numerical methods in the literature, evaluates their performance, and then compare their characteristics. Fig. 2 outlines the overall classification of numerical methods for CFS shear walls under lateral loads, in this study.

3. Macro models for simulation of CFS framed walls

The computational effort, i.e. time and cost, is a key issue in the modelling of large CFS structures, because of their nonlinear behaviour, various instabilities, and relatively large deformations. The computational effort of an FE analysis is considered to be relative to the cubic of the size of a problem [34]. Therefore, the development of efficient macro models for accurate model reduction has recently become a major objective of simulation and modelling. Such models are developed to keep the balance between the required accuracy and efficiency.

For simulation of steel structures, in order to represent the real pre- and post-buckling behaviour, different macro modelling techniques have been developed by researchers. Line element models, with beam or truss elements, are the most widely adopted approaches in analyses of steel shear wall structures (hot-rolled steel shear walls by bracing or steel plate) [38]. To simulate the behaviour of steel shear walls, these approaches employ line type element methods such as multi-angle strip model [39], cross-strip model [40], multi strip model [41], modified strip model [42], combined strip model [43], and equivalent brace model [41] as macro models. While equivalent brace method can be used for both shear walls by bracing and steel plates, other methods are only applicable for shear walls with steel sheets. Fig. 3 schematically depicts these six macro modelling strategies graphs, which are mainly used for simulations of various types of conventional hot rolled shear wall structures.

With regard to CFS framed walls, four classes of macro models have been proposed in the literature to simulate the existing design and construction considerations. These four methods, which are generally developed for numerical modelling of CFS framed walls under lateral loading, are (1) equivalent brace method, (2) equivalent spring method, (3) fastener based method, and (4) effective strip method. Fig. 4 schematically depicts these four macro modelling methods' graphs. The following sections discuss the methods and applications in the literature.

3.1. Equivalent brace method

In this method, the sheathing plate/braces as well as the screws are represented by a single equivalent diagonal brace, whose stiffness is equal to the stiffness of the infill sheathing/brace and screws. This stiffness is derived from experiments. The main advantage of the equivalent brace model lays in the reduced modelling effort and computation time. However, this method is unable to characterize the distributed forces applied by the sheathing on the boundary studs.

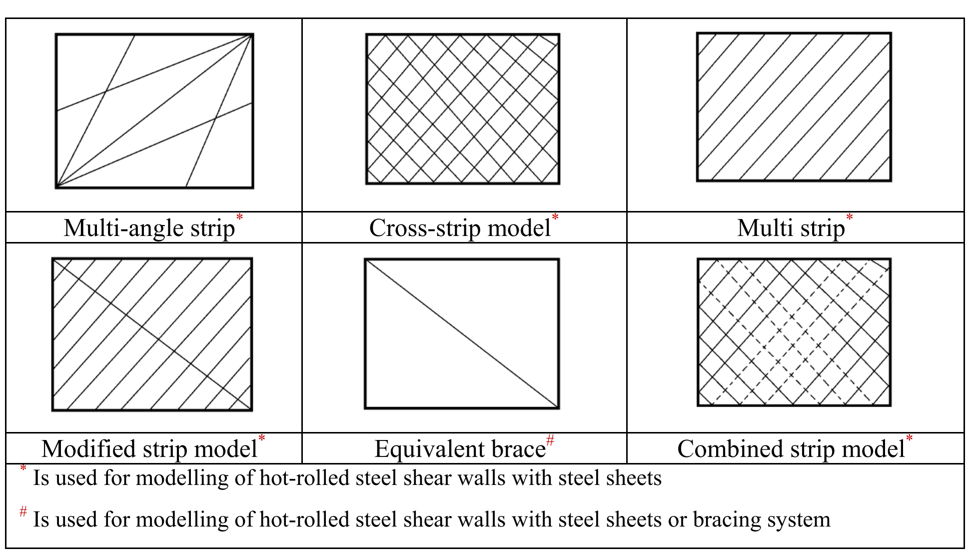


Fig. 3. Some of the available macro models for simulation of steel shear wall structures.

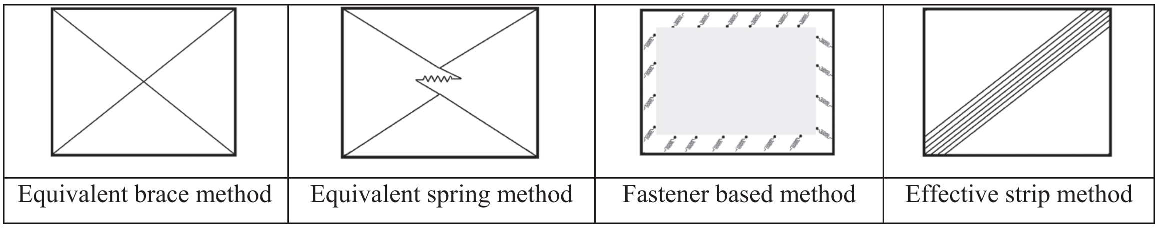


Fig. 4. Macro methods techniques used for modelling of CFS framed walls under lateral loads.

One of the first macro modelling of CFS structures using equivalent brace method was carried out by Gad [44], in which a simplified model of a house was developed and verified against experimental results. The nonlinear time-history dynamic analysis program Ruaumoko [45] and the Stewart [46] degrading hysteresis model were selected for this modelling and dynamic analyses. They assessed the interaction between out of plan veneer walls and frame and verified their model by a modal analysis where the mode shapes and the natural frequencies matched the experimental results. Fig. 5 shows the macro model and the comparison between experimental and numerical results.

Using Ruaumoko program, the seismic force resisting system of two representative buildings was analysed by Boudreault et al. [47,48] in order to assess the performance of the shear wall panels under earthquake loading. In these studies, the Stewart hysteresis model based on the experimental results (Fig. 6) was verified, then two and three storey strap braced structural models were established in order to simulate the oriented strand board (OSB) sheathed shear wall. The gap between the upper and lower walls was created to represent a floor of the two storey model, which is shown in Fig. 6.

Based on the experimental tests on shear walls, a numerical equivalent brace model for hysteresis behaviour of wall panels was created and employed in 3D dynamic nonlinear analysis of CFS framed

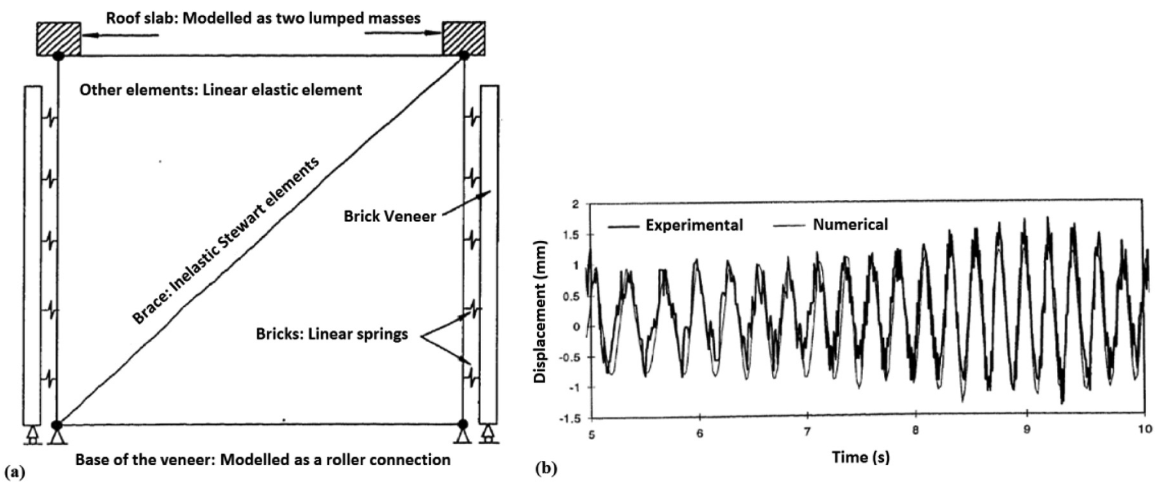


Fig. 5. (a) Equivalent brace model of the tested house, (b) experimental and numerical top of the frame displacement at resonance for SSW input [44].

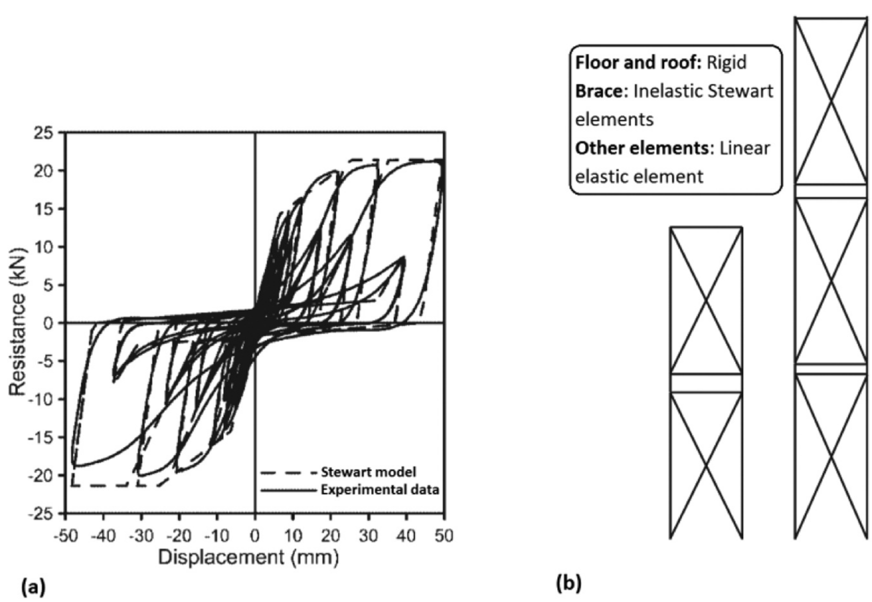


Fig. 6. (a) Resistance versus displacement curve for Stewart model and test data, (b) two and three storey shear wall models [47,48].

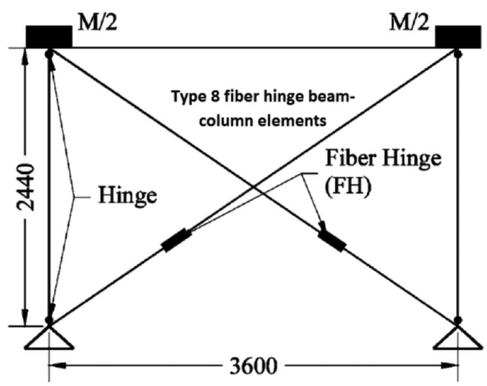


Fig. 7. Wall-panel simulation with equivalent bracing [49].

buildings by Fulop and Dubina [49–51]. A tri-linear model, based on Drain-3DX [52] computer code, was utilized with the full nonlinear model. They hinged all column ends in the model, and assumed that the frame itself is a mechanism not contributing to load bearing capacity (Fig. 7). Their model can consider most of the important features of the hysteresis behaviour and can be implemented in more complex structural systems.

Foutch and Lee [53] simulated two, four and six-storey prototype CFS buildings with gypsum wall under seismic loading. The Drain-2DX [54] program was used for nonlinear time history analyses. They indicated that the inelastic behaviour of the numerical model comes from the truss elements with gap properties for braces and at their ends for columns. Each modelling elements of Drain-2DX and lumped mass position for building are shown in Table 1 and Fig. 8a. The authors had to choose the strength degradation rate of the wall conservatively for the modelling of the gypsum wall, because test results at large drifts were not available. Fig. 8b also demonstrates that the macro model can successfully represent the hysteresis result of the test.

Foutch et al. [55] studied the dynamic behaviour of a two storey, one bay frame under large seismic motions to assess if commonly used numerical models are capable of predicting the measured motion of the structure with an adequate accuracy. In that study, macro model similar to method of Foutch and Lee [53] was introduced in Drain-2DX for dynamic analysis of the specimen, and the results were compared with the results of the shake-table test.

An extensive numerical study by equivalent brace method was developed and then modified by Shamim et al. [56–60] based on the response captured from single and double storey steel sheathed CFS framed shear wall tests. They proposed the macro method using OpenSees [61] software for modelling steel sheathed CFS framed shear wall specimens under dynamic loading. The modelling was carried out

Table 1
Element used in numerical modelling by Foutch and Lee [53].

Element	Element assigned in Drain-2DX	Detail
Stud	Elastic beam element using Plastic Hinge Beam-Column	–
Track	Plastic Hinge Beam-Column Element (type 02)	To represent a rigid element with high stiffness and moment resistance
Brace	Inelastic Truss Bar Element (Type 01) in conjunction with truss element with gap property	To consider pinched model in the truss element
Sheathing	Horizontal spring Element (Type 10, Elasticity Code 4)	–
End of studs	Inelastic rotational spring elements Element (Type 04)	Plastic hinges was expressed to express the moment capacity of the column

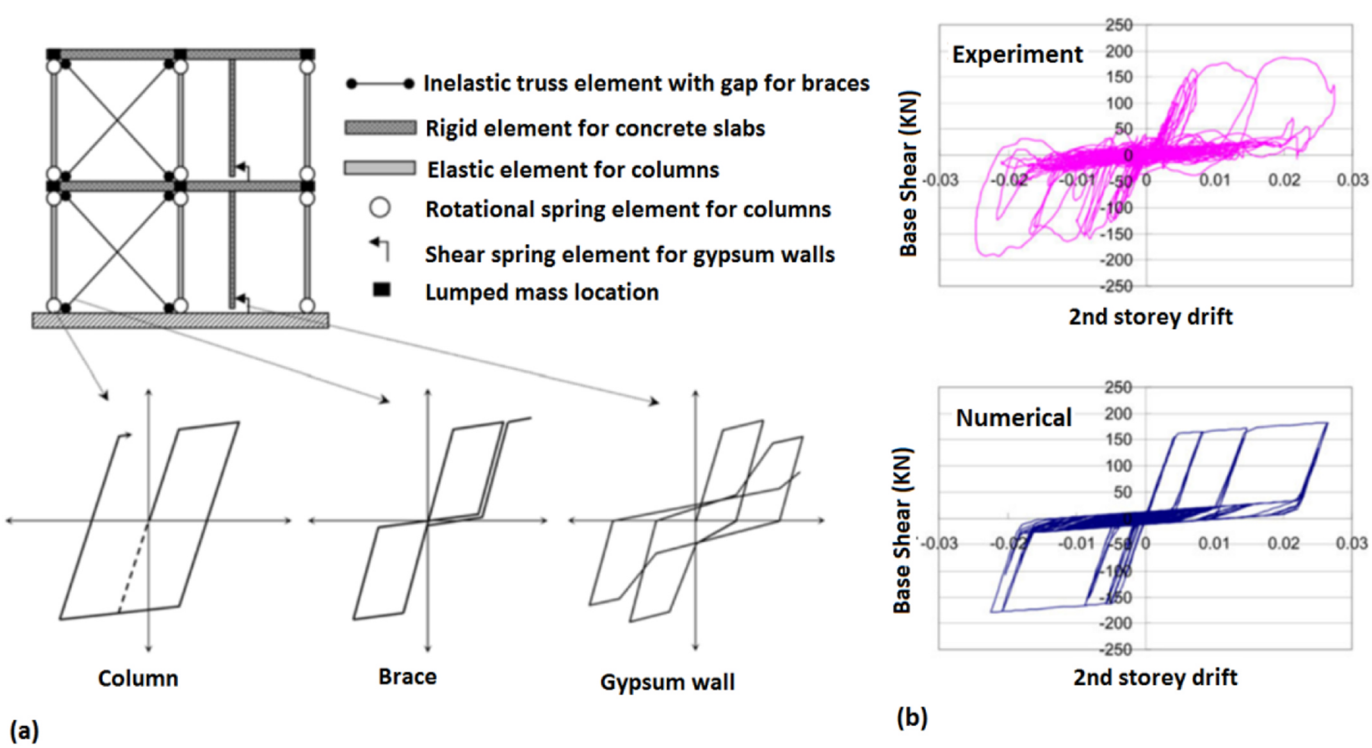


Fig. 8. (a) Macro model for 2-storey CFS Building, (b) comparison of numerical and experimental results [53].

Table 2
Element used in numerical modelling by Shamim et al. [56–60].

Element	Element assigned in OpenSees	Detail
Stud	Elastic beam-column elements	–
Track	Rigid beam-column elements	–
Strap and Sheathing	Inelastic Pinching04 truss members	–
Hold-down	Linear elastic uplift spring elements	To determine the lateral displacement of the wall associated with elongation of anchor rod
Stud-track connection	Elastic rotational spring elements for corners	Represents the in-plane flexural stiffness of the bare frame without sheathing
Floor	Elastic truss elements	–
P-delta effect (fictitious column)	Rigid beam-column element with co-rotational coordinate transformation capability	–
P-delta effect (linking the fictitious column to the CFS frame)	Rigid truss element linking	–
Seismic mass	Lumped at each storey level	Representing the supporting columns and seismic weight

before and after dynamic testing of shear walls [57]. Table 2 and Fig. 9a show the elements used for the model. Their findings indicated that the final lateral displacement of the walls calculated with the model could be affected by three main factors: Shear force of the frame, flexural displacement of the frame, and uplift displacement of each wall section's rigid rotation due to elongation in anchor rod. Fig. 9 also displays the comparison of the test and numerical results in that study.

In another study by Shamim et al. [62], an archetype building developed and calibrated based on the findings of their previous work. Fig. 10 illustrates the components of the macro model utilized for the CFS wall of a double storey archetype building used in that study.

Lu et al. [63] proposed a numerical model for sheathed walls that can be used to evaluate the effect of gypsum on behaviour of a strap braced building. The modelling strategy as well as experimental test data for verifying their model were based on Shamim [56]. The OpenSees macro model with its components, as well as a comparison between the numerical model and the results of the corresponding cyclic test for the strap-braced wall frame are presented in Fig. 11.

An improved equivalent bracing model of CFS shear walls with concrete-filled rectangular steel tube column (as reinforced end studs) was proposed by Wang et al. [64] in order to evaluate seismic behaviour of mid-rise CFS structures. Two different types of modelling, and the elements used in their model are shown in Fig. 12 and Table 3 respectively. They concluded that by taking end stud's compression buckling and beam-column joint's behaviour into account, the results for model 1 are much closer to the experimental results (the error is below 9%) than model 2 with the maximum relative error up to 24%.

A number of comprehensive numerical studies on seismic response of a two-storey CFS framed building with OSB sheathed shear walls (was formally a part of the Network for Earthquake Engineering Simulation research program, or in short CFS-NEES) were performed by Leng et al. [65–69]. The authors' earlier works on modelling the seismic response of CFS-framed buildings normally relied on simplifications to minimise computational cost [67,68]; however, significant differences were reported between numerical and experimental results. Therefore, they developed higher fidelity models that offered dependable

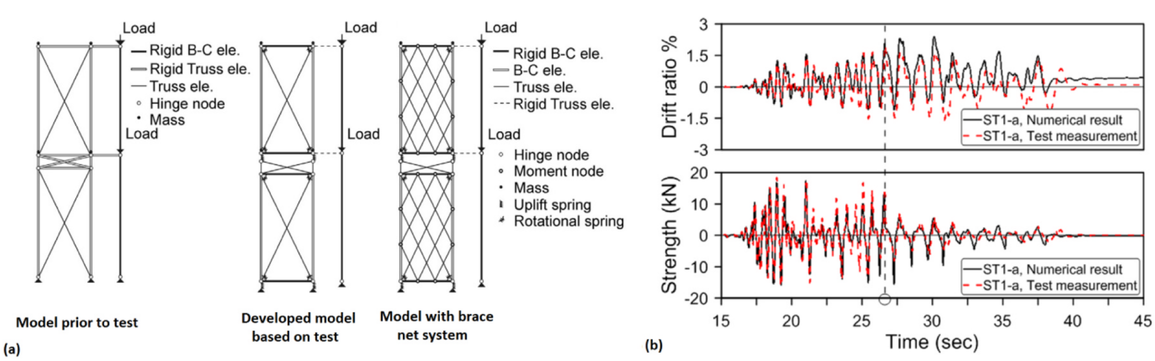


Fig. 9. (a) Numerical models in OpenSees, (b) comparison of numerical and experimental results [60].

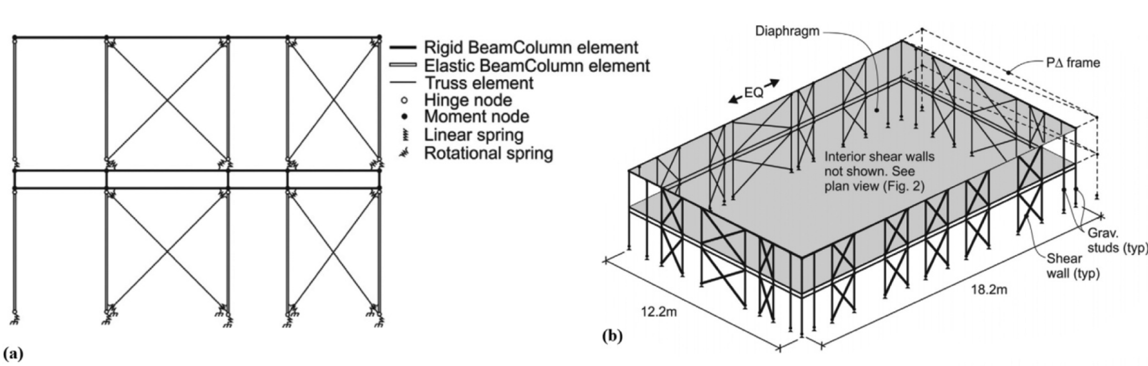


Fig. 10. Schematic representation: (a) office building model used without P–Δ framing, (b) residential building [62].

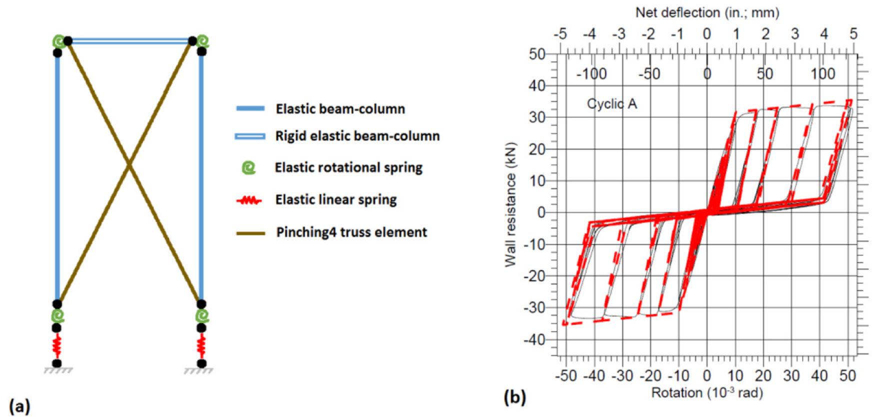


Fig. 11. (a) Macro model of the shear walls in OpenSees, (b) comparison between the numerical model and cyclic test [63].

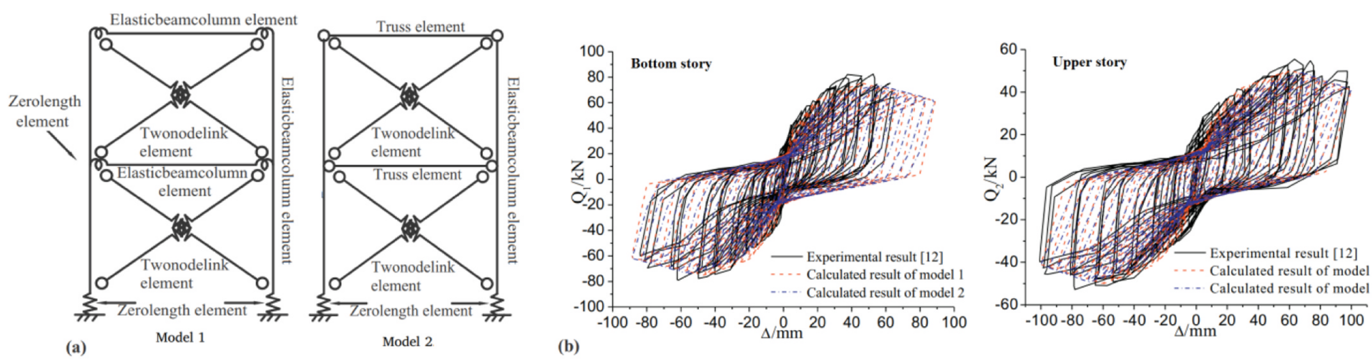


Fig. 12. (a) Macro models of mid-rise CFS shear wall, (b) load-displacement curves of the double-storey specimen [64].

Table 3
Element used in numerical modelling by Wang et al. [64].

Element	Element assigned in OpenSees	Detail
Stud	Elastic beam-column elements	–
Track	Elastic beam-column elements and truss element rigid bar	Based on the rigid diaphragm assumption
Strap and Sheathing	Nonlinear spring elements with Pinching4	–
Hold-down	Axial spring elements	–
Stud-track connection	Rotational spring elements for chord stud	Due to the effective connection between end studs and foundation by hold-downs
End studs	Axial spring along Y-direction	To account the possible buckling at the bottom of columns

prediction of CFS-framed building response under seismic loads [65,69]. A remarkable feature of their simulated shear walls was the subdivision of the sheathing board into subpanels. It was mentioned that in actual framed system a number of intermediate members were connected to the shear wall, including the ledger, window and door headers. Hence, the secondary load paths were allowed in the model by subdividing the shear. Fig. 13 shows the models with and without subpanels, and the simulated CFS-NEES building. The method given in that study provides comprehensive details of how the building attains its beneficial performance.

The CFS-NEES building was redesigned by Yu et al. [70–73] to apply the new corrugated steel sheathing shear walls to the system. A numerical model was developed and seismic performance was evaluated through incremental dynamic analysis using a methodology proposed by FEMA [74]. They employed rigid connection method for their macro modelling, because their linear static analysis results showed that the diagonal bracing stiffness is much greater than the small moment stiffness of the stud to-track connection. In addition, they indicated that using two spring for hold-downs (One spring uses a Pinching4 and other spring uses an elastic-perfectly plastic gap material, with the gap close to zero and a very large stiffness in compression) is more reliable in simulations. Fig. 14 shows the macro model of shear wall and the comparison between experimental and numerical results.

In recent years, a European research project named ELISSA [75] was conducted in order to study the seismic performance of CFS shear walls sheathed with nailed gypsum based panels. In this project, Fiorino et al. [76] developed a macro model for shear wall panels with an ability to model their nonlinear hysteresis characteristic and possessing the capability of being employed in the collapse simulations of whole building. The elements implemented in their numerical study are described in Table 4. Fig. 15 also shows the model developed in the study, as well as the force vs displacement response curves of both numerical simulations and experimental results for a long and short shear wall. They reported that numerical models were able to capture the experimental hysteresis response in terms of final shape and peak locations as well as dissipating energy similar to the experimental tests.

After modelling and verifying the single CFS strap braced stud walls, complete 3D models of archetypes, were created in another study by Fiorino et al. [77]. In this modelling strategy, floor elements (composite floor, joist and racks) are assumed to be rigid elements, due to their large stiffness. Moment releases in this model were established between studs and rigid floors in order to avoid the transfer of moments from

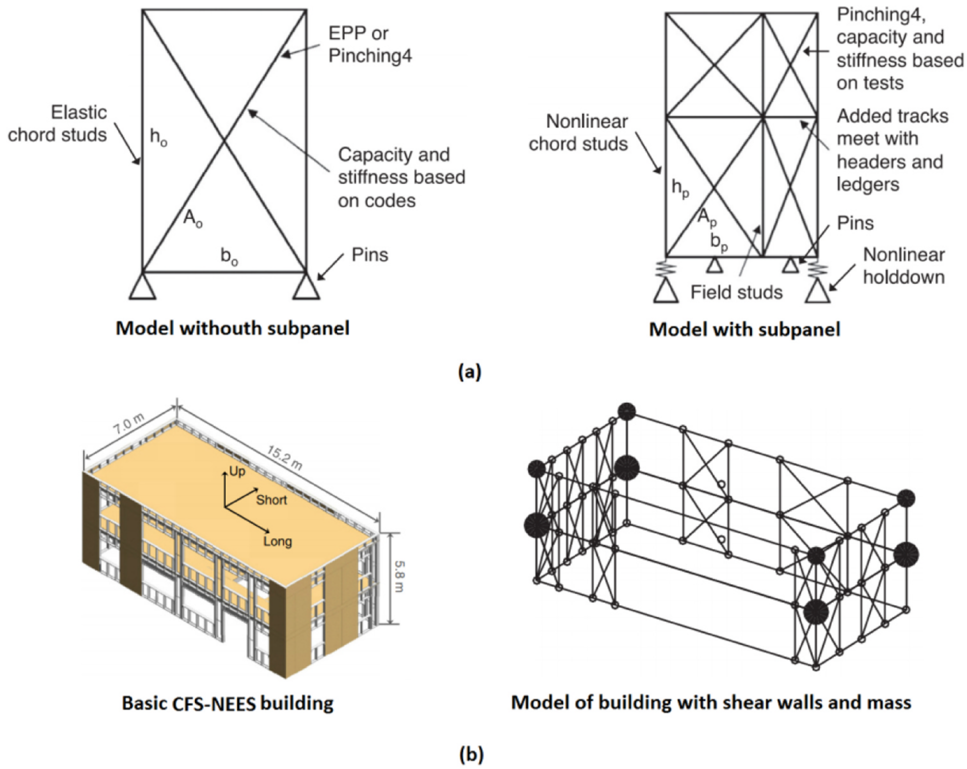


Fig. 13. (a) Comparison of modelling strategies, (b) CFS-NEES building and macro model [65,69].

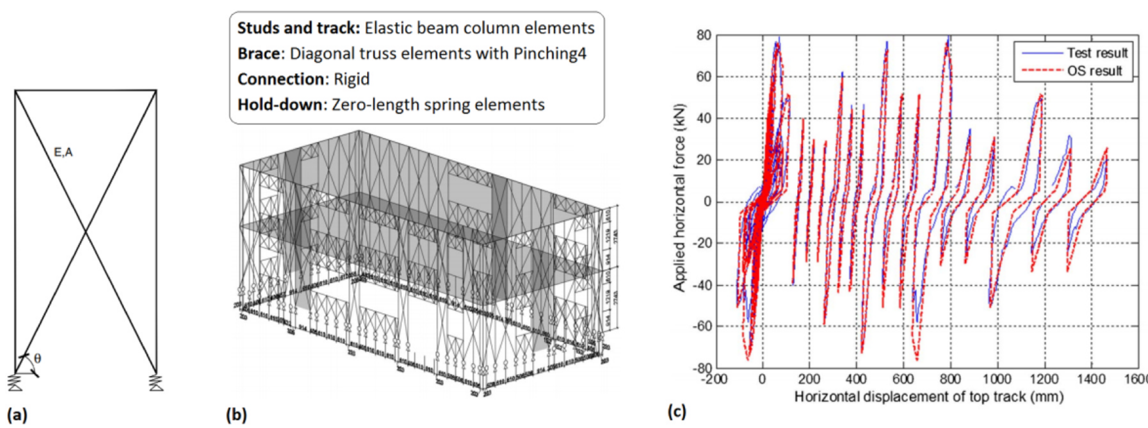


Fig. 14. (a) Macro model of shear wall, (b) developed model for a two storey building, (c) comparison between the experimental and numerical results [70].

Table 4
Element used in numerical modelling by Fiorino et al. [76].

Element	Element assigned in OpenSees	Detail
Stud	Elastic beam-column elements with MinMax material in conjunction with uniaxial elastic material	To model the failure of chord stud due to tension or global buckling
Track	Rigid horizontal displacement constraint	The effect of rigid diaphragm was combined by constraining the horizontal displacements
Brace	Truss element with Pinchinng4 material	–
Hold-down	Zerolength elements with Elastic Multi Linear material in conjunction with MinMax material	In order to consider tensile failure of anchors

floor. The gravity load was applied on the chord studs of walls based on their tributary areas. In addition, seismic mass was utilized at the four corners of building. Fig. 16 schematically illustrates a 2D illustration of a braced bay in two storey residential building designed for low intensity seismic loads.

A numerical macro model was developed in OpenSees by Macillo et al. [78,79], which is able to simulate the dynamic response of the entire building, while considering the effects of non-structural systems. The authors indicated that their model is able to predict the response of the first floor with good accuracy, whilst the prediction of the

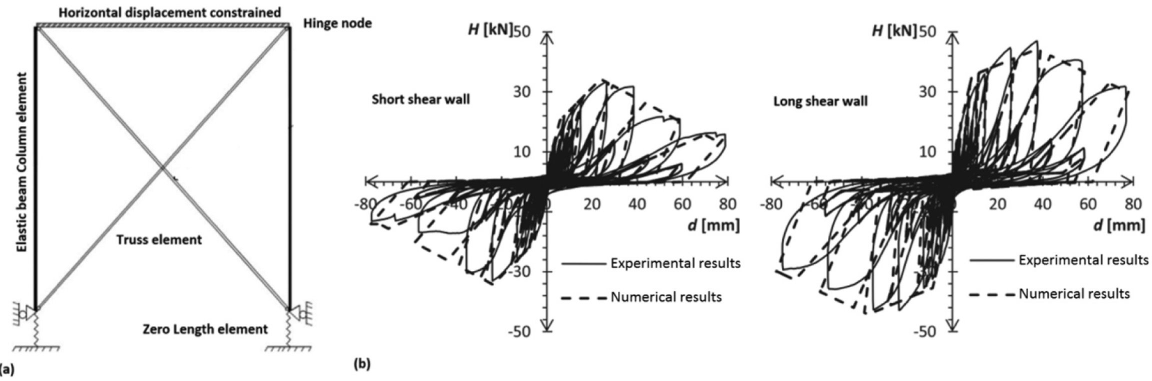


Fig. 15. (a) Macro model for CFS sheathed-braced shear walls, (b) comparison of load- displacement curves between numerical model and test [76].

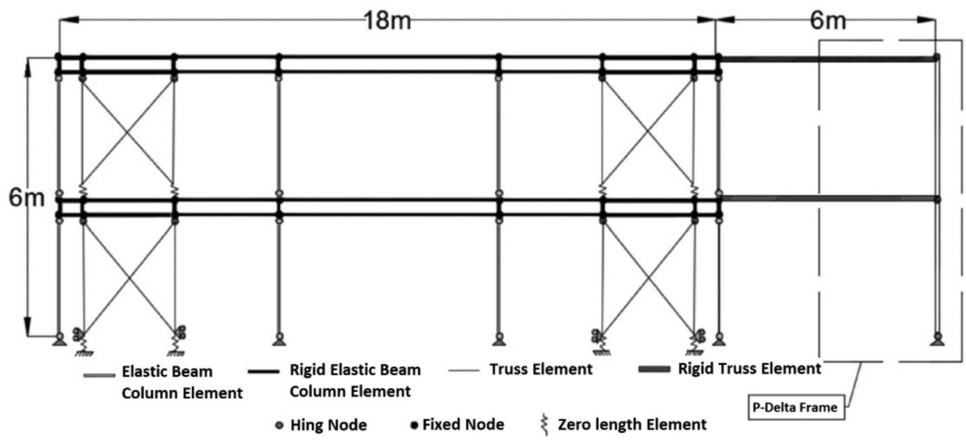


Fig. 16. 2D Schematic of a braced bay in residential building [77].

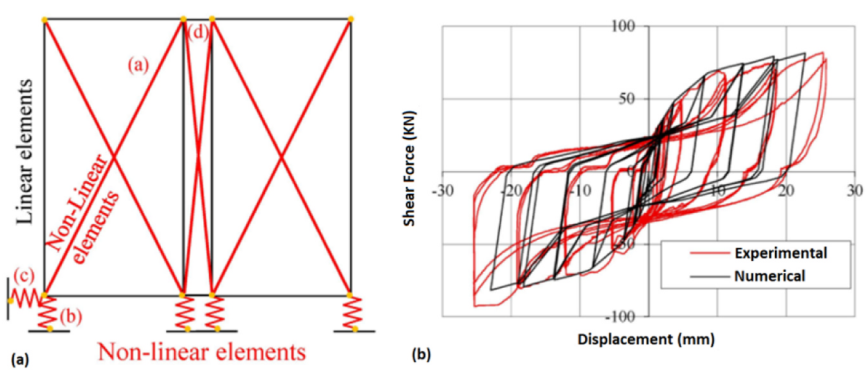


Fig. 17. (a) macro model of a one storey frame, (b) comparison of numerical and experimental results [80].

behaviour of the second storey is not so precise. In another similar macro model, by Scotta et al. [80], the dynamic non-linear behaviour of buildings is evaluated. The numerical results were then used to assess the proper behaviour factor value, according to the European seismic codes. Fig. 17 shows the macro model of the one storey wall system in their study with its elements, and also the comparison of numerical and experimental results.

Table 5 briefly summarises some basic information about the numerical research works on lateral behaviour of CFS framed shear wall structures by equivalent brace method, presented in this section.

3.2. Equivalent spring method

Equivalent spring method is another macro modelling strategy for the simulation of CFS framed shear wall structures under lateral loads. Similar to the equivalent brace method, in this approach the sheathing plate/braces as well as the screws are simulated by a single equivalent spring, in which the overall stiffness and strength of the sheathing/brace and screws are equal to the stiffness and strength of the spring. The lateral stiffness and strength are derived directly from the spring element implemented in shear wall.

For a successful macro model, it is required to verify the design procedure and the R values using dynamic analyses or dynamic tests. To address this need, fourteen structures (4, 6 & 7 storeys) were designed and modelled By Morello [81] employing two different software packages: Ruaumoko and SapWood [82]. With SapWood, they employed a multi-dimensional model of a structure, in which a number of walls were placed throughout the multi-storey building. This technique offers a considerably less complicated model, while still describing the behaviour of a single shear wall under dynamic loading. Fig. 18 and

Table 5
Summary of the macro model studies by equivalent brace method.

Author, reference	Year	Software	Employed hysteresis model	Specimen modelled	Brace or sheathing system
Gad, [44]	1997	Ruaumoko	Stewart	1 storey domestic house	Brick veneer
Boudreault et al., [47,48]	2005 2007			2 and 3 storey frame	OSB sheathing
Fulop and Dubina, [49–51]	2002 2004 2008	Drain-3DX	Trilinear	Single wall panel	OSB, gypsum and corrugated steel sheathing
Foutch and Lee, [53]	2010	Drain-2DX	Bilinear with gap property and trilinear	2, 4 and 6 storey building	Brace and gypsum sheathing
Foutch et al., [55]	2007		Bilinear with gap property	2 storey building	Strap brace
Shamim, [56]	2013	OpenSees	Pinching4	1 and 2 storey wall panel and 2,4,5 storey office and residential building	Steel sheathing
Shamim et al., [57–59]	2011, 2012, 2013			1 and 2 storey wall panel	
Shamim et al., [60]	2013			1 and 2 storey wall panel and 2 storey office building	
Shamim et al., [62]	2015			2,4,5 storey office and residential building	
Lu et al., [63]	2015			Single wall panel	Gypsum sheathing
Wang et al., [64]	2017			1 and 2 storey wall panel	concrete-filled rectangular steel tube column as reinforced end studs, and double layer wallboard
Leng et al., [65–69]	2012, 2013, 2015, 2016, 2017			2 storey building (CFS-NEES building)	Gypsum and OSB sheathing
Yu et al., [70,72]	2014, 2017			2 storey building	Corrugated steel sheathing
Yu et al., [71]	2017			2,3,4, 5-storey hotel and office building	
Yu et al., [73]	2018			Single wall panel and 2 storey building	Non-perforated corrugated steel sheathing and corrugated steel sheathing with vertical slits.
Fiorino et al., [76]	2018			Single wall panel	Nailed gypsum sheathing
Fiorino et al., [77]	2017			1,2 3,4 residential and office building	Strap brace
Macillo et al., [78]	2018			2 storey residential building	Gypsum-based panels
Macillo et al., [79]	2018			Single wall panel	Strap brace
Scotta et al., [80]	2015			Single wall panel and 3 storey building	OSB sheathing and external techno-prene plaster-infilled slab

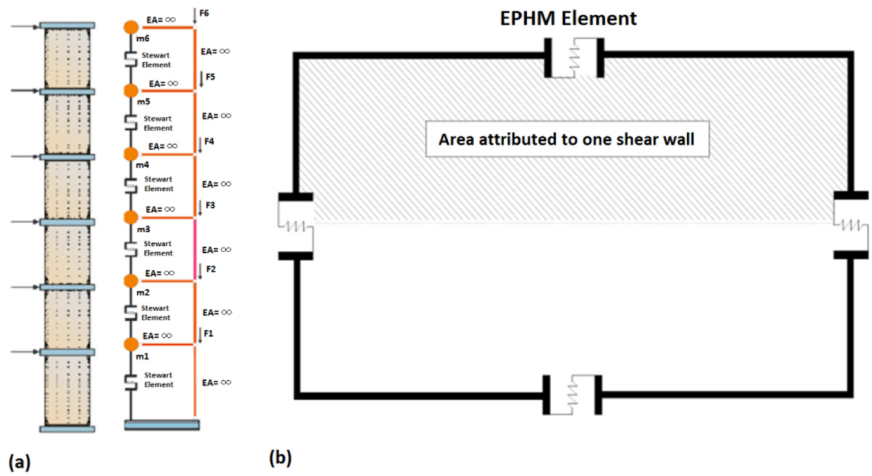


Fig. 18. (a) Shear wall in Ruaumoko for 6 storey building (b) plan of shear building model in SapWood [81].

Table 6
Element used in numerical modelling by Morello [81].

Element	Element assigned in software	Detail
Connection between floors	Spring element with Stewart hysteresis model	To dissipate the seismic energy
Lumped mass	–	Assigned to the node at each floor
P-delta effect	Infinitely stiff column	Its lateral displacement was set to be the same as the corresponding nodes on the shear wall
Walls in SapWood	Spring elements	EPHM hysteresis parameters was used

Table 6 show the elements used in both software and schematically depict the models. They reported that the structures modelled in SapWood fails at lower scaling factors compared to the structures modelled in Ruaumoko, because SapWood considers strength degradation. Generally, a more conservative result was provided with the SapWood models in their study.

Similar to the modelling technique of Morello [81], dynamic analysis of multi-storey structures was carried out by Balh [83] in order to validate the recommended R-values and to determine height limits provided in building regulations. The building was simulated as a stick model in Ruaumoko without considering the exact location of each shear wall. Non-linear dynamic analysis of a multi-storey structure, designed using the AISI S-213 [84] provisions and the NBCC [85], was performed by Comeau et al. [86] and Velchev [87]. The aim was to verify their capacity based design approach, the R_d and R_o values and the building height limit. They compared the six storey stick model and full brace/chord stud model, in order to confirm the application of the stick models for the analyses. They indicated that the simpler (stick) model significantly reduces the needed computational time. Fig. 19 displays both simple and complex models.

Another macro FE modelling technique for CFS shear wall panels was proposed by Bourahla et al. [88]. The strategy was based on substituting the entire panel by a nonlinear spring element connected to rigid body elements transmitting the forces to the end studs resisting tension and the compression. A number of vibrations testing on a recently constructed five storey building were used in their study to validate the initial elastic stiffness of the wall panels. Fig. 20 displays the macro model proposed in their study as well as the real building used for verification of the numerical method.

A numerical study of the seismic behaviour of an innovative light-gauge CFS mid-rise building, designed using direct displacement design method, was presented by Dao and Lindt [89,90]. This advanced system comprised open panel, floor trusses, V-braced panels, columns and connections between components. First the numerical method by experimental data of a wall panel was verified and then a five-storey example building was examined. The panels in the building were modelled by a hysteresis spring (Folz and Filiatrault hysteresis model [91]), and the columns were modelled by beam elements. The second-order effects, i.e., the P-2-D effect, as well as stiffness and strength degradation were also involved in the analysis. Fig. 21 shows a comparison between experimental and numerical results of a wall panel, as well as the developed morel for a mid-rise CFS building.

Shahi et al. [92,93] presented an incremental dynamic analysis on CFS fibre cement board (FCB) shear walls, which comprises a systematic application of non-linear time-history analysis to implement correlations between the damage state of the structure with the severity of earthquake ground shaking. To that end, SapWood computer program was employed for the analysis of equivalent single degree-of-freedom (SDOF) systems.

Kechidi and Bourahla [94] proposed a smooth hysteresis model for lateral behaviour of CFS framed structures that considers stiffness and strength degradation as well as pinching effects. They implemented that model in OpenSees software, as user-defined uniaxial materials named CFSWSWP and CFSSSWP for CFS-wood and CFS-steel sheathed shear wall panels, respectively. The elements used for this macro modelling approach, and a comparison between experimental data by Balh [83] and numerical results for both walls with wood and steel sheathing are shown in Fig. 22.

In another study by the same authors [95], a probabilistic seismic behaviour and risk assessment of CFS sheathed shear wall panel structures was carried out, where a series of 12 building structures, were designed for two seismic intensity levels. To model their nonlinear behaviour, the structures were simulated adopting abovementioned developed model. In addition, a seismic design strategy for CFS structures utilizing sheathed shear wall panels was proposed by Kechidi et al. [96] in accordance with the framework of the Eurocodes and then nonlinear static and incremental dynamic analyses were carried out on 54 CFS frames. First, they verified their model by experimental data (Fig. 23a) obtained from a single frame, and then used the data to develop a macro model of a higher storey frames. The schematic model of a two storey frame and the elements employed for modelling in the study, are given in Fig. 23b and Table 7. It can be seen that the

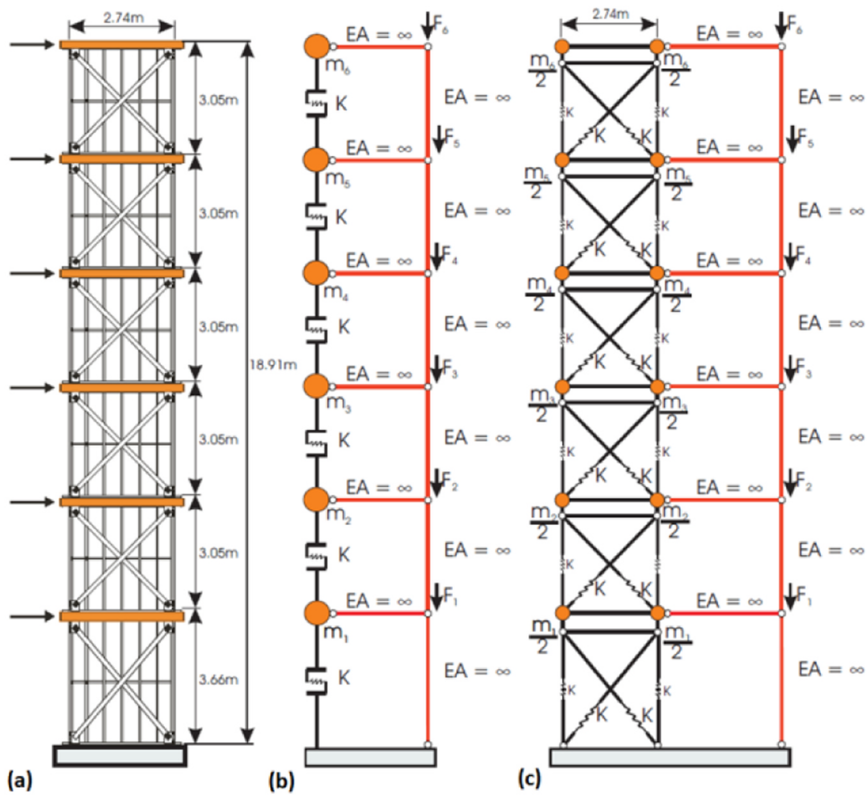


Fig. 19. (a) Six-storey shear wall, (b) macro model, (c) full brace-chord stud model (complex) [87].

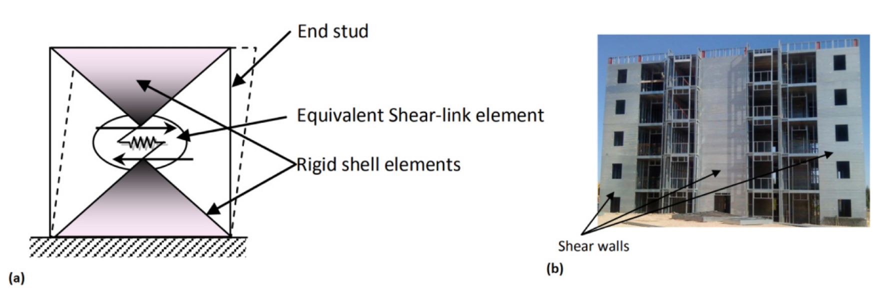


Fig. 20. (a) Macro model of frame (b) real 5 storey building for verification [88].

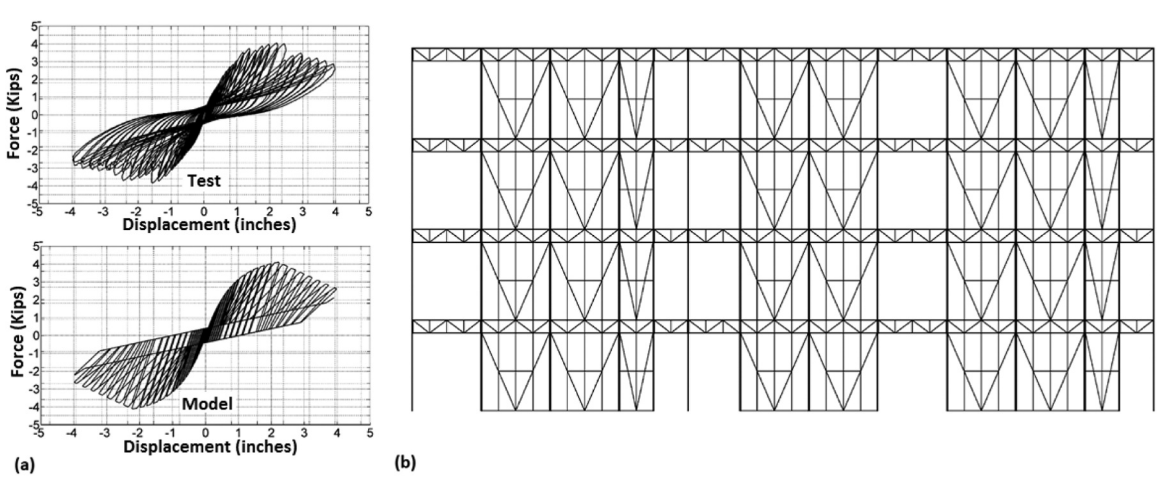


Fig. 21. (a) Comparison of numerical model and experimental data, (b) developed mid-rise building [90].

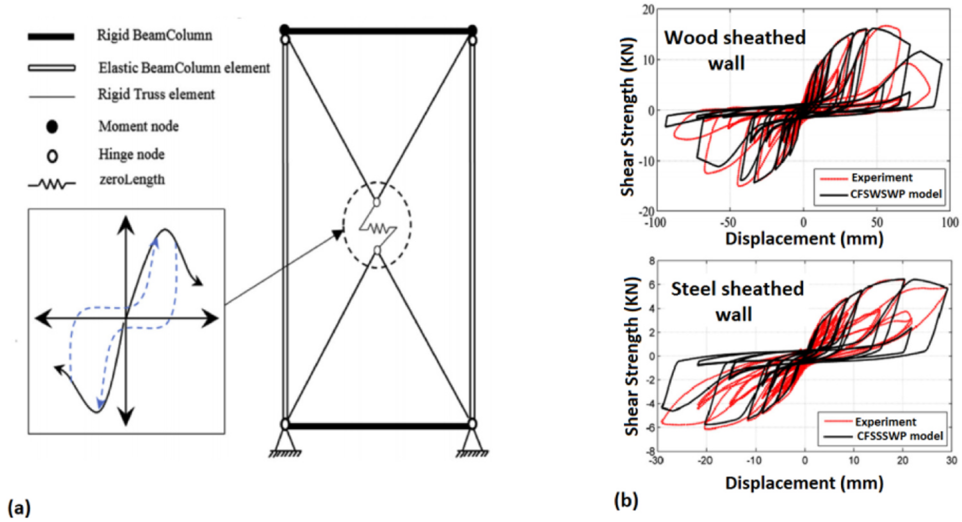


Fig. 22. (a) Macro model of a one storey frame; (b) comparison between model and experiment [94].

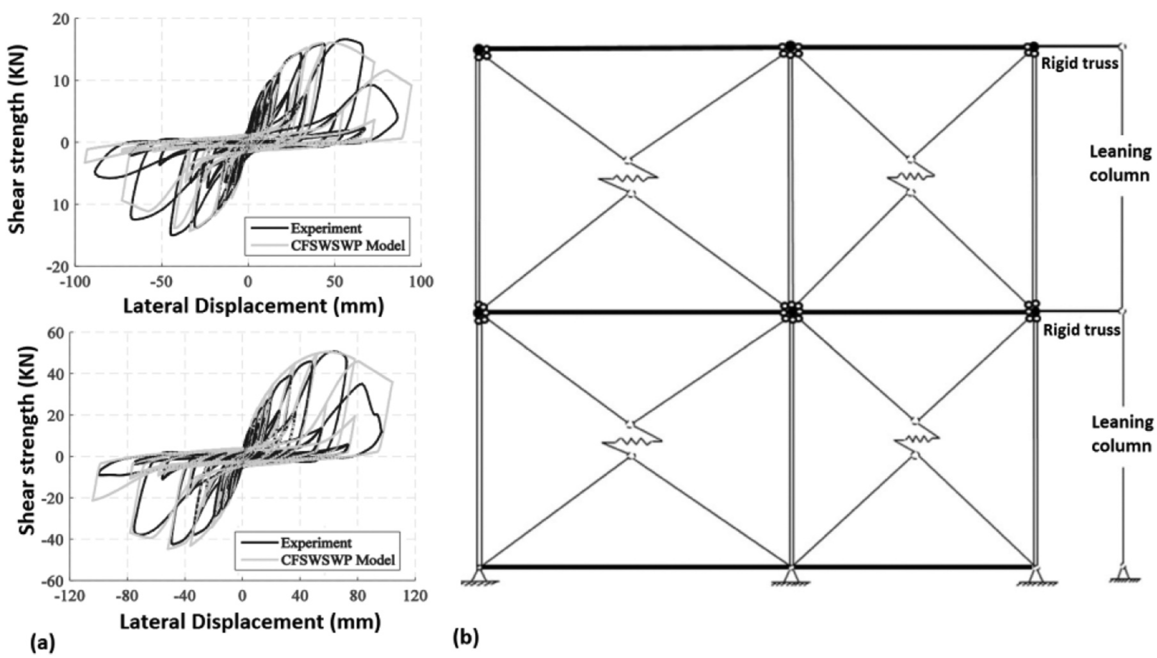


Fig. 23. (a) Verification of the single wall by experimental data (b) Macro model of a two storey frame [96].

Table 7
Element used in numerical modelling by Kechidi et al. [96].

Element	Element assigned in OpenSees	Detail
Stud	Elastic beam-column elements	–
Track	Rigid beam column element	A multipoint constraint is employed to slave the horizontal DOF at each floor level to model a rigid diaphragm
Framing end	Modelling as pin (hinge node)	To prevent any resistance to lateral loads
Connection of leaning column and the wall	Rigid truss elements	Are hinged around the wall
Seismic mass	–	Is uniformly distributed at the top corners of each wall
Bearing and partition walls	Rigid truss element	Leaning column to the CFS frame
Connection to springs	Rigid truss element	–
Sheathing	Zerolength element	–

continuity of chord studs along the height of the structure is not taken into account in their macro models. In their model, it is also noted that the gravity load resisting system has to be prevented from contributing to the lateral stiffness, while considering P-delta effects.

Zeynalian et al. [97] also evaluated the seismic behaviour of CFS-FCB shear walls by nonlinear incremental dynamic analyses of multi-storey CFS structures. A modelling approach similar to Morello's method [81] was utilized in OpenSees software using pinching04

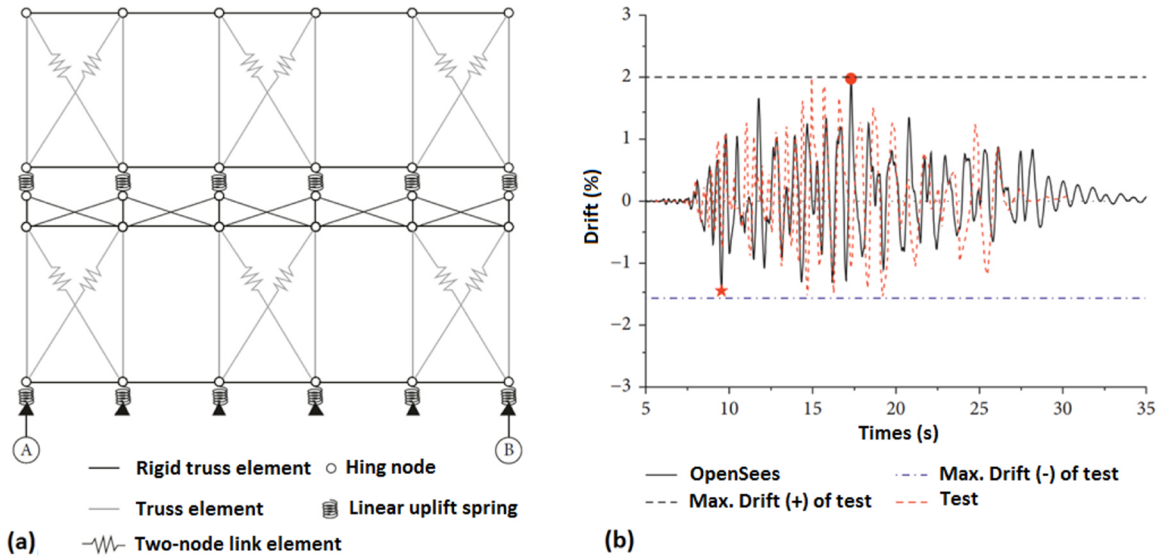


Fig. 24. (a) Macro model of the frame, (b) comparison of model results and test data for specimen ST1 [98].

Table 8
Element used in numerical modelling by Jiang and Ye [98].

Element	Element assigned in OpenSees	Detail
Stud	Elastic truss	–
Track	Rigid truss	–
Framing end	Simplified hinge nodes	Because there is no bending moment transmitted in these connections
Floor and roof	Rigid planes	Planes are connected with the hinge nodes
Hold-down	Linear springs	To consider uplift behaviours of the anchor rods and hold downs
Sheathing	Two-node link elements with Pinching04 material	–

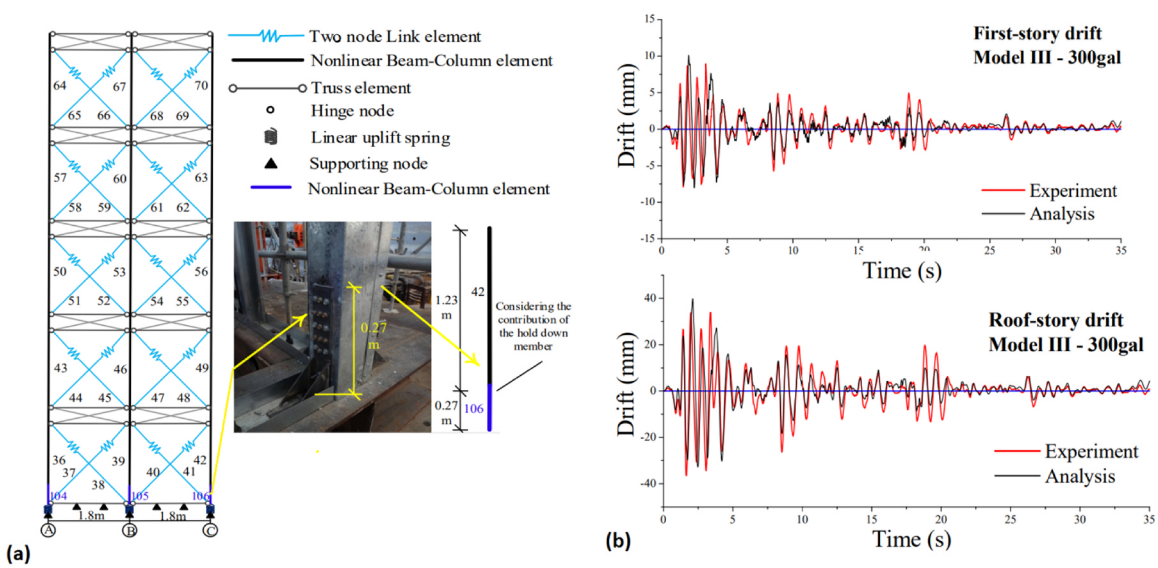


Fig. 25. (a) Macro model for the shaking table test specimen, (b) comparison of numerical and experimental data [99].

element for nonlinear dynamic time history analysis of a FCB wall. In another study, macro models for four 2-storey steel-sheathed CFS framed buildings were provided by Jiang and Ye [98] based on shaking table tests on steel-sheathed CFS walls using OpenSees software. The elements utilized for their macro modelling are presented in Fig. 24a and Table 8. They established a group of fragility curves for CFS buildings after validation of their proposed model, as shown in Fig. 24b.

They also numerically developed a CFS building's model from low-rise to mid-rise, made by a new type of CFS composite shear wall system [99]. The simplified model utilized in that study is shown in Fig. 25a. The rigid diaphragm approach was employed to simulate the composite floor system in order to improve the computational efficiency of the macro model. They compared the numerical results with experimental data of a five-story CFS 1:2 scaled composite shear wall, and reported a reasonable agreement between results. The comparison of numerical and experimental results is illustrated in Fig. 25b.

A summary of basic information on the numerical studies on the lateral behaviour of CFS framed shear wall structures, which were

Table 9
Summary of the macro model studies by equivalent spring method.

Author, reference	Year	Software	Employed hysteresis model	Specimen modelled	Brace or sheathing system
Velchev, [87]	2008	Ruaumoko	Bi-linear with slackness hysteresis	2,4 and 5-storey building	Strap brace
Morello, [81]	2009	SapWood Ruaumoko	Stewart and EPHM	4, 6 and 7-storey building	Wood and gypsum sheathing
Comeau, [86]	2010	Ruaumoko	Bi-linear with slackness hysteresis	2,4,6 and 7-storey building	Strap brace
Balh, [83]	2010		Stewart	4-storey building	Steel sheathing
Bourahla et al., [88]	2012	Sap2000	Pivot	5 storey building	Gypsum or wood sheathing
Dao and Lindt, [89,90]	2012, 2013	NG	Folz and Filiatrault	4 and 5 storey building	V-braced panels
Shahi, [92]	2015	SapWood		Single wall panel	Fibre cement boards
Shahi et al., [93]	2017			Single wall panel and Typical domestic houses	
Kechidi et al., [94]	2016	OpenSees	Pinching04	Single wall panel	Steel and wood sheathing
Kechidi et al., [95,96]	2017a,b			2,4,5 storey building	Wood sheathing
Zeynalian et al., [97]	2018			1,2 and 3 storey building	Fibre cement board
Jiang and Ye, [98]	2018			Two storey building	Steel sheathing
Jiang and Ye, [99]	2018			Five storey building	Gypsum wall board

presented in this section and employed equivalent spring method is provided in Table 9.

3.3. Fastener based method

In the fastener based modelling approach, each fastener (mainly screw) is characterised by a non-linear, radially-symmetric spring element. The material properties of the fastener element are specified from experimental tests of sheathing-to-stud connections. A softening backbone curve, pinching, and loading and unloading parameters are included in the fastener material model. The CFS-sheathing connections highly affect the load-deformation curves of the CFS shear walls. In fact, combined behaviour of connections, frame and sheathing are responsible for the total shear resistance. The interaction between fasteners and sheathing is especially important because first, sheathing-to-steel fastener response is the main reason of shear wall nonlinearity, and second, there is high variation in this fastener response.

Buonopane et al. and Bian et al. [100–106] comprehensively employed this macro modelling technique in order to evaluate CFS framed structures under lateral loading. They simulated full-scale shear walls of several widths with various construction details relevant to the ledger track, gypsum board, vertical and horizontal seams, and number and thickness of field studs. The numerical results were compared to the full-scale shear wall tests in terms of load–displacement behaviour, initial stiffness, lateral strength, drift at failure, and energy dissipation. The results were then compared to specification-based strengths and displacements. The modelling elements and configuration of fastener based model, employed in most of their studies, are presented in Table 10 and Fig. 26a. In these studies, each OSB or gypsum board panel was modelled as a separate rigid body. The rigid panel assumption seems not to be appropriate for steel sheathing material, which withstands considerable deformation within the panel and smaller deformations surrounding the fasteners. Fig. 26(b) depicts the comparison of numerical and experimental results for two models with OSB and gypsum sheathing. The results obtained from the fastener based method were consistent with the test result, but failed at a slightly reduced strength.

In addition, a model was proposed by Padilla-Llano [107] to

Table 10
Element used in numerical modelling by Buonopane et al. and Bian et al. [100–106].

Element	Element assigned in OpenSees	Detail
Stud Track	Linear elastic, displacement-based beam elements	–
Ledger track	Linear elastic beam–column elements along its centreline	Connected to the chord studs using a rigid link that transfers only vertical forces
Hold-down	Uniaxial spring elements	Active in the vertical direction only, horizontal degree-of-freedom is restrained.
Sheathing	Rigid body (RigidDiaphragm in OpenSees)	To assume that deformation in the sheathing occurs locally around the fasteners. It does not include global shear deformation of the sheathing.
Stud to track connection	Semi-rigid rotational springs	Allow for semi-rigid connections
Fastener	Zero-length element (CoupledZeroLength in OpenSees) with uniaxial force-deformation behaviour and Pinching4 material	Symmetric in the plane of the sheathing

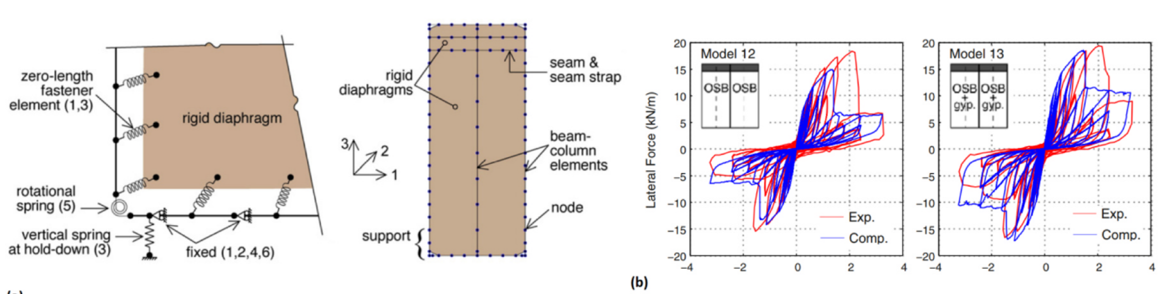


Fig. 26. (a) Macro model of a one storey frame (b) load–displacement response for shear walls [104].

Table 11
Element used in numerical modelling by Padilla-Llano [107].

Element	Element assigned in OpenSees	Detail
Stud Track	Nonlinear beam-column using asympiching behaviour	–
Shear anchors	–	Fixing the horizontal degree of freedom at two of the track nodes next to the hold-downs
Hold-down	Elastic zero Length springs	Low stiffness for tension and high stiffness in compression to simulated the contact with the foundation.
Sheathing	ShellMITC4 element	To accommodate any deformations the sheathing can experience
Contact with foundation	Springs with large stiffness in compression and close to zero stiffness in tension	Allowing uplift of the track nodes
Fastener	Nonlinear Coupled Zero Length element with pinching4	Can provide the flexibility needed for this type of connection and eases the formulation of a model

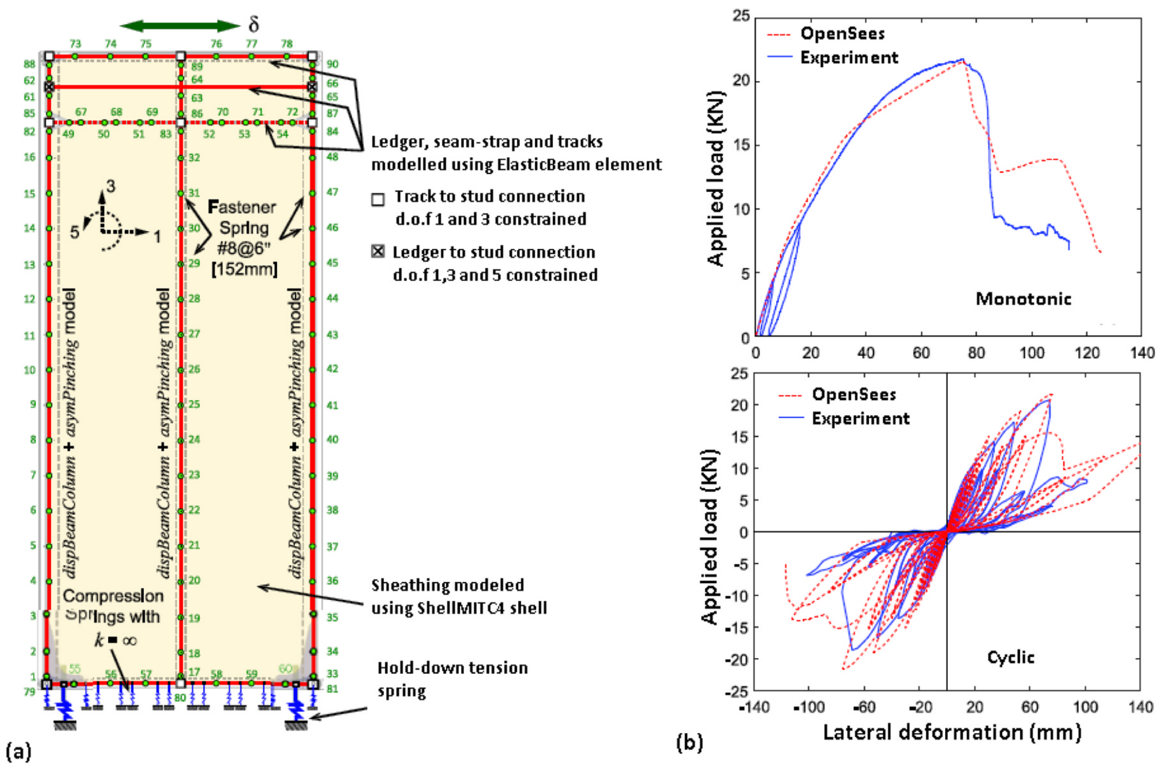


Fig. 27. (a) Macro model of shear wall, (b) comparison of numerical and experimental results [107].

Table 12
Summary of the macro model studies by fastener based method.

Author, reference	Year	Software	Employed hysteresis model	Specimen modelled	Brace or sheathing system
Bian et al., [100,101,105,106]	2014, 2015, 2017a,b	OpenSees	Pinching4	Single wall panel	Gypsum and OSB sheathing
Bian et al., [102]	2015				OSB sheathing
Buonopane et al., [103,104]	2014, 2015				Gypsum and OSB sheathing
Padilla-Llano et al., [107]	2015				OSB sheathing

indicate failure mechanism due to the development of local buckling on the chord studs. To study the effects of the vertical member slenderness on the response of a shear wall, they assigned a specific value to the slenderness of vertical framing members and employed asymPinching model for studs to consider local buckling effects. Table 11 summarises the elements implemented in their study. The macro models of a wall, as well as the comparison between experimental and numerical results are illustrated in Fig. 27.

A summary of numerical studies using fastener-based method on the lateral behaviour of CFS framed shear wall structures is given in Table 12.

3.4. Effective strip method

The effective strip method was theoretically developed by Yanagi and Yu [108], and is generally employed for CFS shear walls with steel sheathing. In this model, it is assumed that a partial width of the steel sheet in the diagonal direction (the effective strip) is engaged in the tension field action to undergo the lateral force applied to the top of the wall. Therefore, the tension force created in the effective strip of the steel sheathing is directly related to the lateral capacity of the wall.

Employing the effective strip method, Santos [109] and Briere et al. [110,111] developed an innovative configuration for CFS walls to

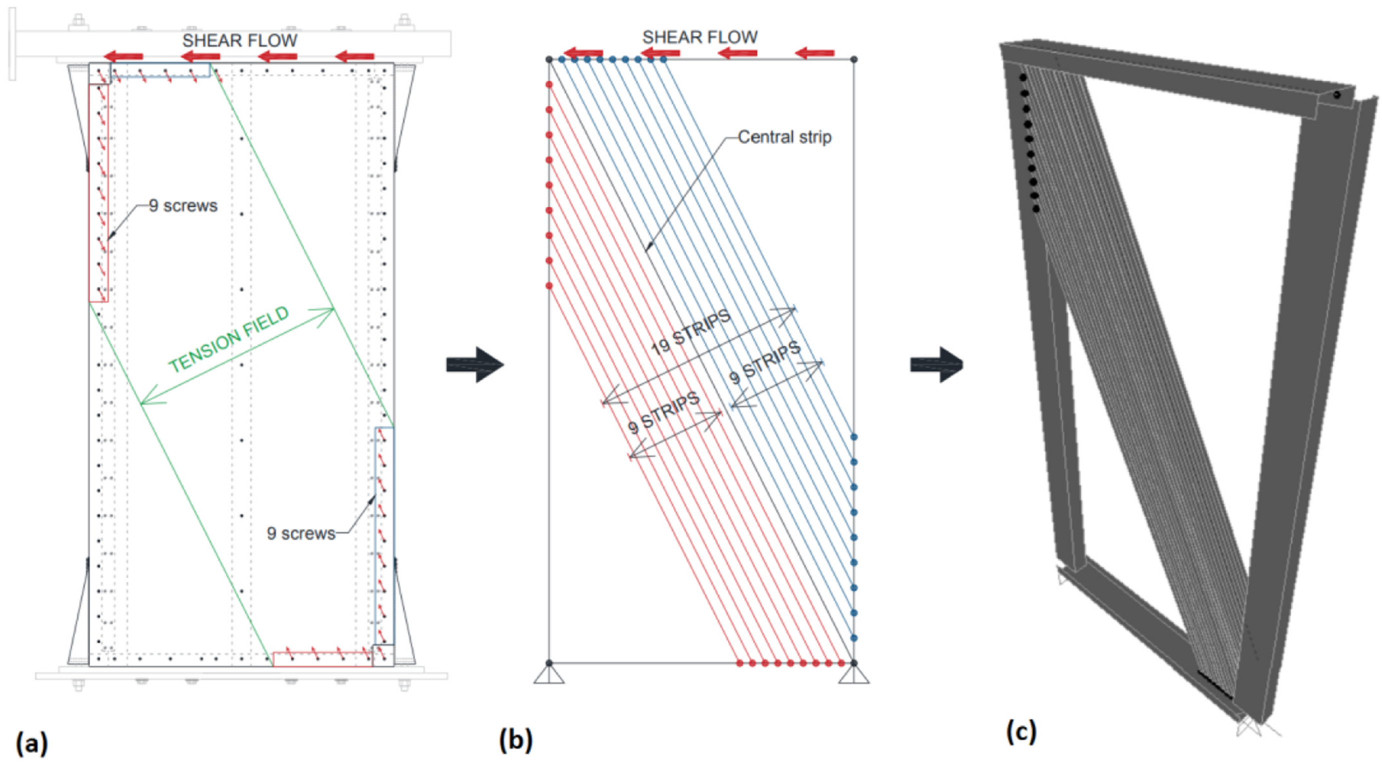


Fig. 28. From experimental specimen to numerical model: a) screws located within the tension field b) equivalent frame elements for the numerical model c) numerical model in Sap2000 [109–111].

Table 13
Summary of the macro model studies by effective strip method.

Author, reference	Year	Software	Employed hysteresis model	Specimen modelled	Brace or sheathing system
Santos, [109] Briere et al., [110,111]	2018 2017, 2018	Sap2000	Not applicable (just ultimate capacity is captured)	Single wall panel	Steel sheathing

address the need for a ductile lateral framing system for mid-rise buildings. In the numerical models, the effective strip method was implemented in Sap2000 program in order to find the ultimate shear capacity of CFS shear walls with steel sheets. The strips were simulated by equivalent strip elements pin-connected to the studs and tracks at the appropriate fastener spacing. In addition, the bottom corners were simply supported, and the framing elements were all pin connected to each other in order to indicate the screw connections between the different framing members. The number of strips used was depended on the number of sheathing connections on the chord stud, located within the tension field. Fig. 28 shows the CFS shear wall from experimental specimen to simplified numerical model in that study. They also calculated the ultimate shear capacity of walls, and reported a good agreement between test data and numerical results.

A summary of numerical studies using effective strip method on the lateral behaviour of CFS framed shear wall structures is presented in Table 13.

3.5. A comparison of hysteresis models used in macro modelling methods

Generally speaking, an appropriately designed CFS framed structure dissipates energy mostly through the inelastic performance of its connections. When a CFS framed structure is subjected to frequent cyclic loading, the generated hysteresis loops are characterised by strength and stiffness deteriorations as well as a pinching effect. Such characteristics, which significantly contribute to the post-elastic behaviour of the system, must be taken into consideration in the dynamic nonlinear analyses. The essential demand to implement such analyses is the availability of a basic model able to simulate as precisely as possible the structure response when exposed to a quasistatic or dynamic loading. Due to the complicated nature of the behaviour, and the difficulties occurred in simulation, many hysteresis models proposed in the literature ignore some (or even a majority) of the key aspects observed in experimental test. Researchers on the other hand, have developed a variety of complex hysteresis models attempting to represent hysteresis behaviour of shear walls as accurately as possible. These models have been mainly used for research purposes and are not commonly utilized for seismic analysis of real structures. Eight hysteresis models, being employed by researchers in the numerical models for the study of CFS framed structures under lateral cyclic loading, are briefly reviewed here in this section, and the characteristics of each model are discussed. Some theoretical research studies have been also carried out in order to calculate the hysteretic behaviour of CFS wall panels [112–115], which are beyond the scope of this paper.

3.5.1. The evolutionary parameter hysteresis model (EPHM)

This model was developed by Pang et al. [116] and can be used for many engineering fields. The EPHM employs a total of seventeen parameters to capture the nonlinear hysteresis behaviour of shear walls. The EPHM is a relatively good choice for peak displacement analyses, when validity in the displacement calculations is needed over the whole range of design hazard levels. The model is able to precisely account the degradation of a CFS wood sheathed shear wall by modifying its loading and unloading paths with evolutionary parameters. The ability of the EPHM to obtain energy dissipation at large displacements also makes it a good option for performance-based design applications, which may consider the performance requirements associated with significant deformation demand.

3.5.2. Stewart model

The Stewart hysteresis model was proposed by Stewart [46] and found to best represent the strength and stiffness characteristics of a steel frame with wood panel shear wall components. The model is a SDOF model and can only examine the overall wall response. Stewart model is commonly in use for CFS wood sheathed shear walls studies and is included in the Ruaumoko [45] inelastic dynamic analysis software package. A series of rules are employed to develop this model, which offers pinching and stiffness degradation but not strength degradation. As reported by some authors [44,47,48,81,83], the main issue with modelling CFS framed structures under lateral load by this model is the lack of strength degradation considerations.

3.5.3. Folz and Filiatrault model

This hysteresis model, which is also used for CFS wood sheathed shear walls, was developed by Folz and Filiatrault [91]. A key characteristic of this model is the lack of a linear part of the load-deformation curve, even at a low displacement level. A total of ten parameters are required to validate this model; parameters that can be captured either from experimental test of shear wall or from the results of an numerical analysis with the CASHEW computer program [117]. The model is added to the dynamic analysis software SAWS (Seismic Analysis of Woodframe Structures) [118] to be used for lateral assessment of CFS wood sheathed framed structures.

3.5.4. Pinching4 model

One of the currently in-use and widely applicable hysteresis models for lateral evaluation of CFS structures with different sheathing materials is Pinching4 model, which was developed by Lowes et al. [119] and is being implemented in OpenSees software. Pinching4 parameters comprise the backbone points in addition to parameters defining the pinched and unloading/re-loading behaviour of the model. The model is able to capture pinching, stiffness and strength degradation depending on damage level, such as unloading stiffness degradation, reloading stiffness degradation, and strength degradation.

3.5.5. Pivot model

This model is developed by Dowell et al. [120] and is generally used for reinforced concrete members; however, it was also used by Bourahla et al. [88] for lateral evaluation of CFS shear walls. Pivot model is able to take into account the strength degradation, effect of axial load and lack of section symmetry and is also implemented in Sap2000 software.

3.5.6. Bi-linear with slackness hysteresis model, tri-linear model and bilinear inelastic with gap hysteresis models

The bi-linear with slackness model can be utilized to represent diagonal braced systems where yield in one direction may stretch the elements leading to slackness in the bracing system [52,54]. The bi-linear inelastic and bi-linear with slackness models, employed in the literature, are unable of considering strength deterioration, due to repeated loading [52,54]. In order to cover this inadequacy, they have been defined based according to the stabilised envelope of the cyclic curve.

Table 14 shows a summary of the abovementioned hysteresis models, in which the strength degradation is captured; and Table 15 show those, in which the strength degradation is not taken in to account.

4. Micro method for simulation of CFS framed walls

Micro models are the most accurate and widely-used tool available for in-depth analysis of the CFS framed structures' behaviour. The main advantage of using micro modelling methods is that all possible failure mechanisms can be captured. Although micro method is a numerical approach, whose results may not be considered as exact answers, it can provide adequately accurate results for most engineering problems. A precise micro model for CFS structures must comprise the basic types of boundary conditions and local failures mechanisms. The accuracy of micro method modelling of CFS framed shear walls under lateral load depends on many parameters such as the geometry, material properties, boundary condition and interactions between components, connections, solver systems and elements. Fig. 29 categorises the parameters that affecting the numerical results of CFS framed shear wall structures under lateral loading, discussed in the literature. Different numerical models have focused on one or some of these parameters to study their role in the CFS walls performance.

In recent years, there has been an extensive growth in application of computer programs, corresponding to micro modelling of different CFS structures under various loading conditions [121–124]. In this section, the micro modelling methods in the literature for simulating the behaviour of CFS framed shear walls are classified based on the computer programs they have employed for modelling. In general, three FE computer programs are being widely used for micro modelling of CFS shear walls in the literature: Abaqus [125], Ansys [126] and Sap2000 [127]. An absolute majority of micro modelling of CFS shear walls under lateral loads in the literature are carried out using one of these three software packages. For the analysis of CFS shear walls, each package possesses its own capabilities and limitations, employs different behavioural models, is affected by different factors, and characterised by different parameters and functions, which will be discussed in this section.

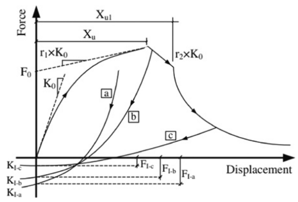
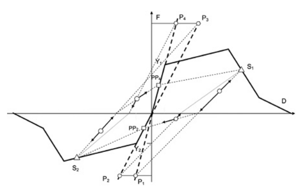
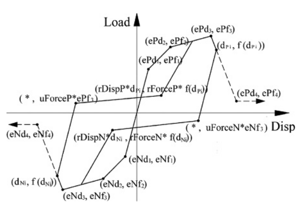
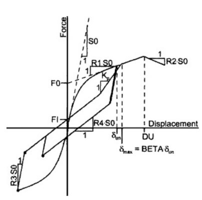
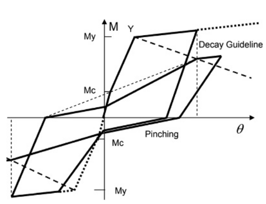
4.1. Micro modelling studies by Abaqus

Abaqus [125] is an advanced and valuable general FE software, which has been used extensively for micro modelling of CFS components in recent years. It is also being employed as a powerful tool for micro modelling of CFS framed shear wall structures under lateral loading through implementing appropriate criterion and parameters to ensure the accuracy of the results.

Using Abaqus computer package, a FE model was developed by Telue and Mahendran [128] to understand the performance of steel wall frames with plaster boards on both sides. They utilized two load steps in the non-linear analyses, one for residual stress and the other for applying lateral load. Parametric studies on the effect of the first screw connection's variations, the effect of plasterboard fastener spacing and thickness was also conducted in that study. Attari et al. [129] simulated CFS shear walls with single and double sided steel sheathing under monotonic loading. The details of the specimens, boundary conditions and materials are precisely modelled based on the experiments. For creating an imperfection to the numerical models, the middle point of the steel sheathing is pushed 10 mm out of plane, prior to lateral pushover analysis. They indicated that their micro model is in good agreement with test results with respect to post buckling response, the peak strength estimation and initial stiffness. Fig. 30 shows the micro model and comparison of numerical and experimental results.

In another study, non-linear FE analyses were employed by Niari et al. [130] in order to examine the seismic performance of steel sheathed CFS shear wall panels. Geometric and material non-linearity were also included in their FE models. They noted that the displacements corresponding to the maximum loading, obtained by FE method, are smaller than experimental data and therefore, the elastic stiffness of numerical models are greater than corresponding experimental specimens. Their numerical method was able of capturing the seismic

Table 14
Hysteresis models used for lateral performance of CFS framed structures (with strength degradation).

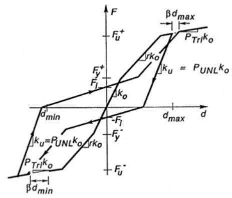
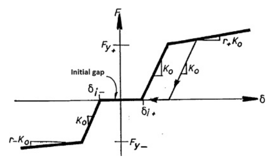
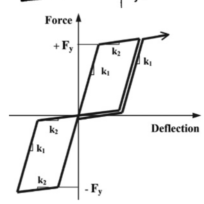
Model name	Model shape	Description	Used by
EPHM		**K0:** Initial stiffness, **F0:** Resistance force parameter of the backbone **r1:** Stiffness ratio parameter of the backbone **Xu:** Displacement corresponding to max. restoring force of the backbone **r2:**Ratio of the degrading backbone stiffness to K0 **Xu1:** Displacement corresponding to the end of linear portion degrading backbone **P1:** Exponential degrading rate parameter of the backbone **FIm and Fir:** Max and min value of residual pinching force **DFIa and DFIb:** Damage index corresponding to the starting/End point of the plateau portion of the FI **PFI and Pr4:** Exponential degrading rate parameter of the FI and KI degrading function **r4r:** Ratio of the residual KI to initial stiffness **β:** Strength degradation parameter	[81]
Pivot		**P_1:** P1 through P4 on the elastic loading lines control the amount of softening in each quadrant **P_2:** Pinching Pivot points PP2 and PP4 fix the degree of pinching following load reversal in each quadrant **Fy, D:** yield resistance, degradation point **αF_y:**resistance of primary pivot points **PP:** resistance of pinching pivot point **α_1 and α_2:**locates the pivot point for unloading to zero from positive and negative force **β_1 and β_2:** locates the pivot point for reverse loading from zero toward positive and negative force	[88]
Pinching 4		**ePf1,ePf2,ePf3,ePf4 and ePd1,ePd2,ePd3,ePd4:** Defining force and deformation points on the positive response envelope **eNf1,eNf2,eNf3,eNf4 and eNd1,eNd2,eNd3,eNd4:**Defining force and deformation points on the negative response envelope **rDispP and rDispN:** Defining the ratio of the deformation at which reloading occurs to the maximum and minimum historic deformation demand **fFoceP and fFoceN:** Defining the ratio of the force at which reloading begins to force corresponding to the maximum and minimum historic deformation demand **uForceP and uForceN:** Defining the ratio of strength developed upon unloading from negative load to the maximum strength developed under monotonic loading **gK1,gK2,gK3,gK4,gKLim** and **gD1,gD2,gD3,gD4,gDLim:** Controlling cyclic degradation model for unloading and reloading stiffness degradation **gF1,gF2,gF3,gF4,gFLim:** Controlling cyclic degradation model for strength degradation	[56–60,62–73,76–80,94–107]
Folzand Filiatrault		**S0:** Initial stiffness of shear wall spring element **F0:**Resistance force parameter of the backbone **F1:** Pinching residual resistance force **R1:** Stiffness ratio parameter of the backbone, **R2:** Ratio of the degrading backbone stiffness to K0 **R3:** Ratio of the unloading path stiffness to K0 **R4:** Ratio of the Pinching load path stiffness to K0 **Du:** Drift corresponding to the maximum restoring force **B (Beta):** Strength degradation parameter for shear wall spring **A (Alpha):** Stiffness degradation parameter for spring element	[89,90,92,93]
Trilinear		**K0:** Initial wall stiffness **Mc:** moment at first linear behaviour **My:** Yield moment	[49–51,53]

behaviour of actual CFS shear wall, when compared numerical and experimental results in terms of shear resistance, stiffness and failure modes. Fig. 31 shows the numerical results compared to experimental data.

A numerical micro modelling study was developed and validated by Borzoo et al. [131], on the CFS shear panels with steel sheathing, in order to evaluate the stiffness, strength and failure mode of walls. In a recent study by Hatami et al. [132], the behaviour of CFS steel sheathed shear walls and their response modification factors were investigated and the accuracy of the numerical method was assessed. A parametric study containing various ranges of wall parameters such as wall height, steel sheet thickness, spacing of screws, and thickness of the frame members was also implemented in that study.

Although it is possible to model the corrugated geometry of steel sheeting in many advanced commercial software packages, the process is time consuming and fairly complex. Dai [133,134] developed a numerical micro method for the structural behaviour of typical CFS walls sheathed by corrugated steel sheets, in which they simplified the corrugated sheet into an equivalent orthotropic flat sheathing board with two elastic moduli for simplifying calculation and analysis. After validation of the numerical method, they adopted different parameters for the equivalent sheet in order to understand their effect on the structural behaviour of a wall panels. In a similar study, Yu et al. [73] performed a series of numerical analyses in order to capture nominal shear strengths for corrugated steel sheathed shear walls. They utilized tie constraints for stud-to-stud and stud-to-track connections as no framing connection

Table 15
Hysteresis models used for lateral performance of CFS framed structures (without strength degradation).

Model name	Parameter	Description	Used by
Steware		**K0:** Initial wall stiffness **Fu:** Ultimate force **Fy:** Yield force **R:** Bi-linear factor beyond yield force **Fi:** Intercept force **PTri:** Tri-linear factor beyond ultimate force **PUNL:** Unloading Stiffness factor **Gap+:** Initial slackness. Positive axis **Gap-:** Initial slackness, negative axis **B (Beta):** Softening factor **A (Alpha):** Reloading or pinch power factor	[44,47,48,81,83]
Bi-linear with slackness		**K0:** Initial wall stiffness **R:** Bi-linear factor beyond yield force **Fy:** Yield force **Δ:** Initial slackness	[86,87]
Bilinear inelastic with gap		**K0:** Initial wall stiffness **Fy:** Yield force	[53,55]

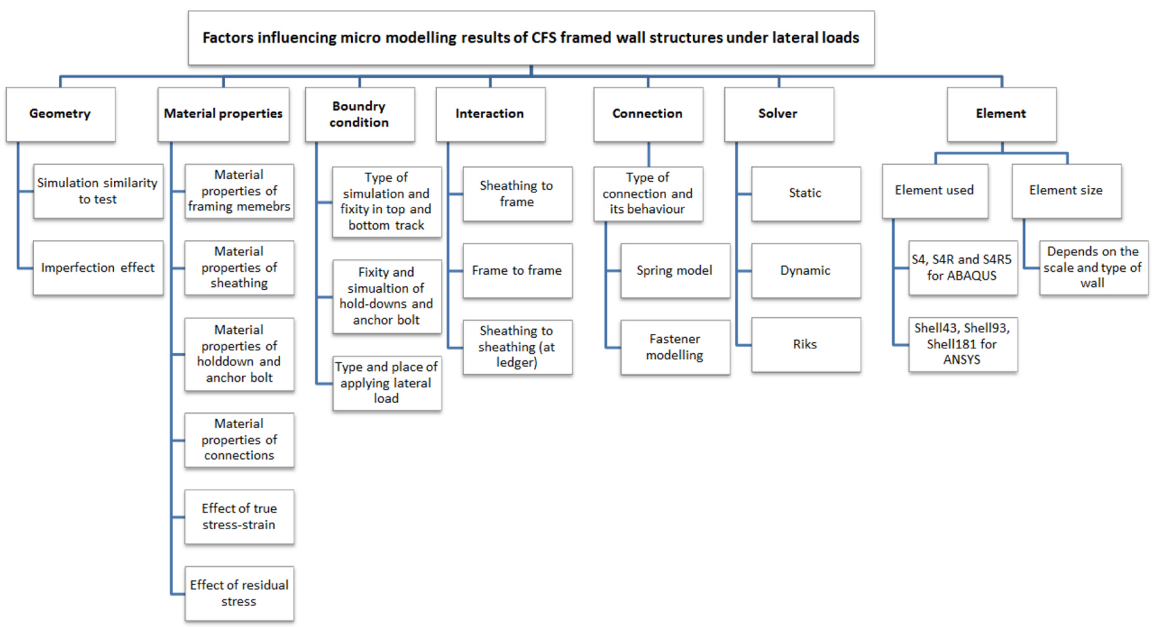


Fig. 29. Factors affecting numerical results of CFS framed shear wall structures under lateral loading.

failure occurred in the tests. The sheathing-to-frame and sheathing-to-sheathing screws were also modelled by Spring2 elements in their study. Modelling strategy and comparison of the load-deformation responses are illustrated in Fig. 32. Although their model was able to match the shear wall behaviour prior to the peak load and the initial stiffness, it was noted that the displacement at the peak load, obtained from the test, varies slightly from those captured by numerical method.

A reliable and detailed micro modelling strategy that can be used for accurate simulation of wood-sheathed CFS shear wall was proposed by Ngo [135]. Their numerical model provided a conservative prediction of the peak load for the specimens only sheathed by OSB, and provided an optimistic prediction for the specimens, in which gypsum board was also included. Fig. 33 shows the micro model of their shear wall as well as a comparison of experimental and numerical results, which shows that the developed numerical models can precisely capture the initial stiffness, but become overly stiffer afterwards. Similar to Ngo's [135] modelling strategies a micro model of CFS shear wall was developed in Abaqus by Bian et al. [102]. The final result was a model, which was similar to OpenSees models in many ways, but included a full and accurate 3D treatment of the framing. They concluded that the OpenSees and Abaqus results agreed well with one another.

Due to some commercial FE software limitations, it is not simple to capture the complete hysteresis behaviour of CFS-sheathing's connections. To achieve a widely applicable modelling protocol for both monotonic and cyclic analysis, Abaqus requires an extension that incorporates complete CFS-sheathing connection hysteresis behaviour. To

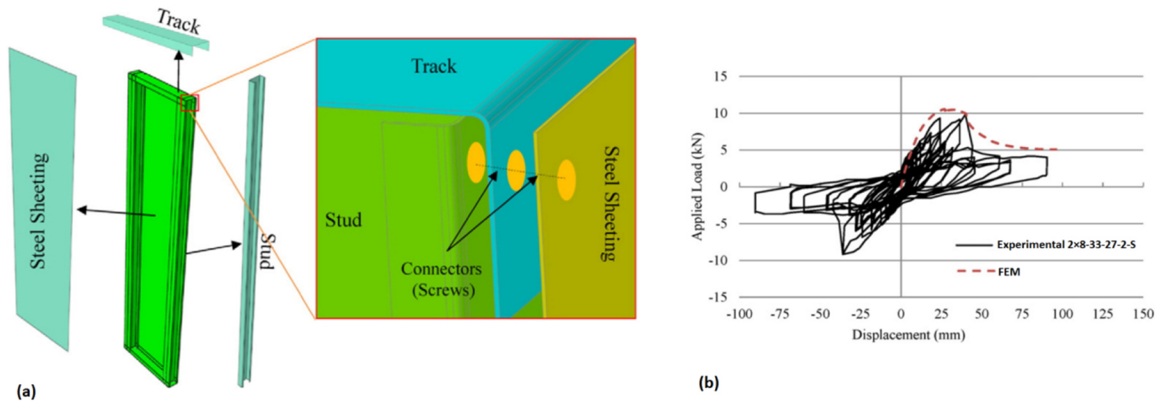


Fig. 30. (a) Simulated model, (b) lateral load displacement response of numerical and experimental [129].

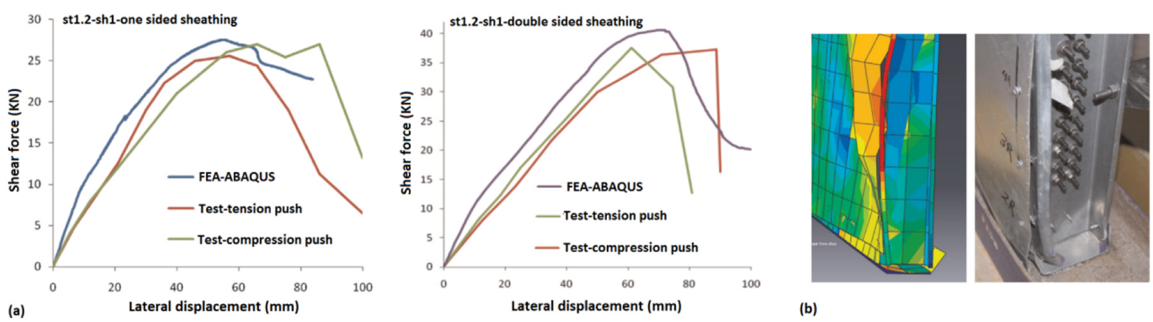


Fig. 31. (a) Comparison of numerical and experimental results, (b) failure mode of CFS shear wall panel [130].

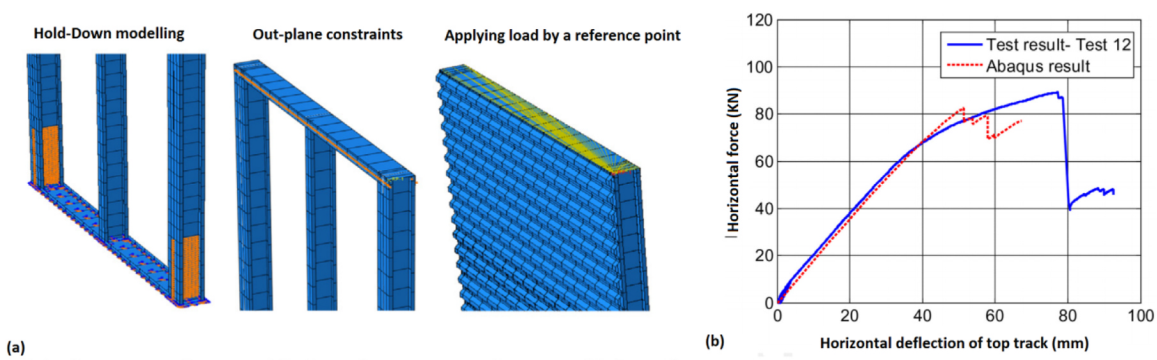


Fig. 32. (a) Modelling strategy, (b) comparison of numerical and experimental results [73].

that end, a comprehensive study was performed by Ding [136] on the modelling of both monotonic and cyclic response of CFS framing screw-fastened connections. An Abaqus user element (UEL) was written and validated for a nonlinear hysteresis model that can simulate pinching and strength and stiffness degradation for CFS screw-fastened connections. In that study, the OpenSees Pinching4 model parameters were implemented in Abaqus and the method was verified by comparing to OpenSees connection simulation results. In addition, unlike Ngo's study [135], which assumed the OSB sheathing as a rigid diagram, Ding [136] accounted the real OSB material strength in order to consider flexural and shear deformation of OSB sheathing boards. Configuration of micro model in that study, comparisons of numerical and experimental results, as well as pinching behaviour captured by Abaqus are shown in Fig. 34.

For further evaluation of the contribution of OSB boards on CFS shear walls, in comparison with a steel-braced and non-braced panel, a series of numerical simulations were conducted by Henriques et al. [137]. They also used the numerical method to assess the impact of additional bracing systems, such as standard diagonal steel strips bracings. In that study, it was assumed that all screw connections and anchorage were fully rigid. They reported that (i) the numerical model is not able to capture the complete behaviour of the panel under lateral loading, when the behaviour is governed by the connections; and (ii) accurate results is obtained, when the connections remain in the elastic range. Mojtabaei et al. [138] also tried to numerically evaluate the seismic behaviour of an innovative CFS moment resisting frame. A micro model was developed by considering the material non-linearity and geometrical imperfections. The ultimate strength, lateral load-displacement behaviour and failure modes estimated by the model were in very good agreement with the test data in that study. The validated model was then employed to evaluate the effects of key design parameters on the lateral load capacity, ultimate displacement, energy dissipation, ductility, and ductility reduction factor of the frame. Fig. 35 shows the model and the numerical results of their study.

The micro modelling studies on the lateral behaviour of CFS framed shear wall structures using Abaqus computer package, in the literature, was reviewed in this section. A summary of the modelling techniques classified based on the “boundary condition”, “interaction and

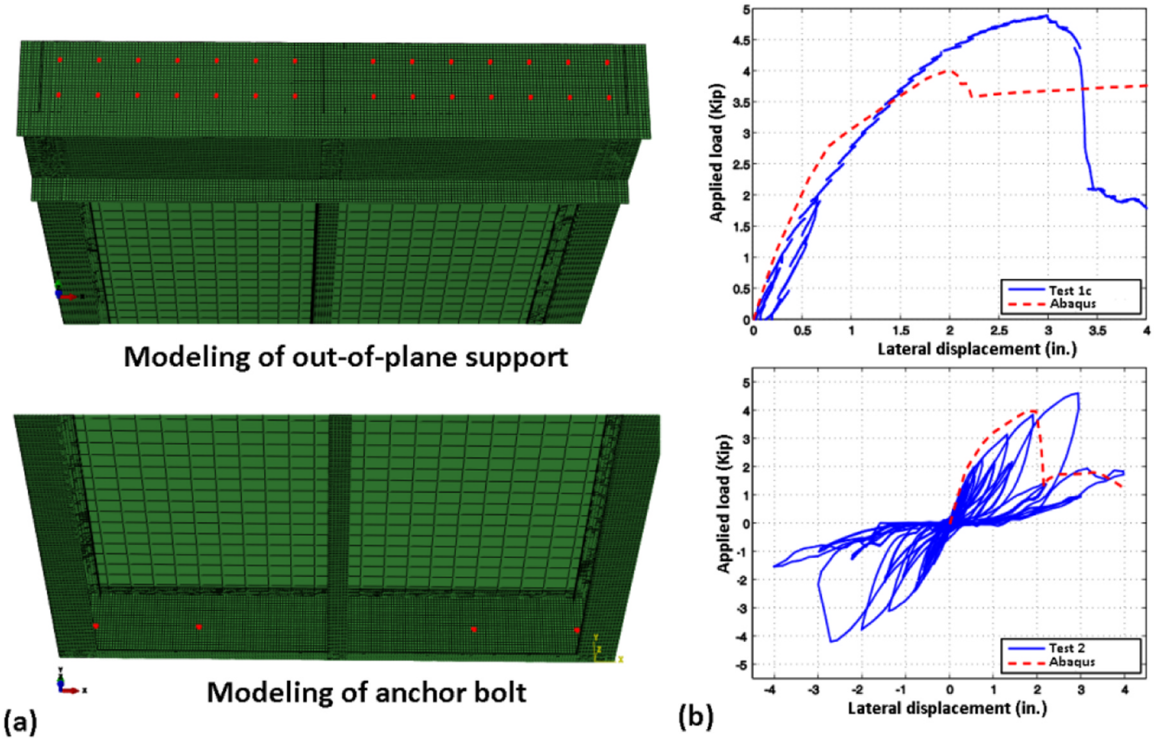


Fig. 33. (a) FE model, (b) nonlinear response of micro models compared with experimental results [135].

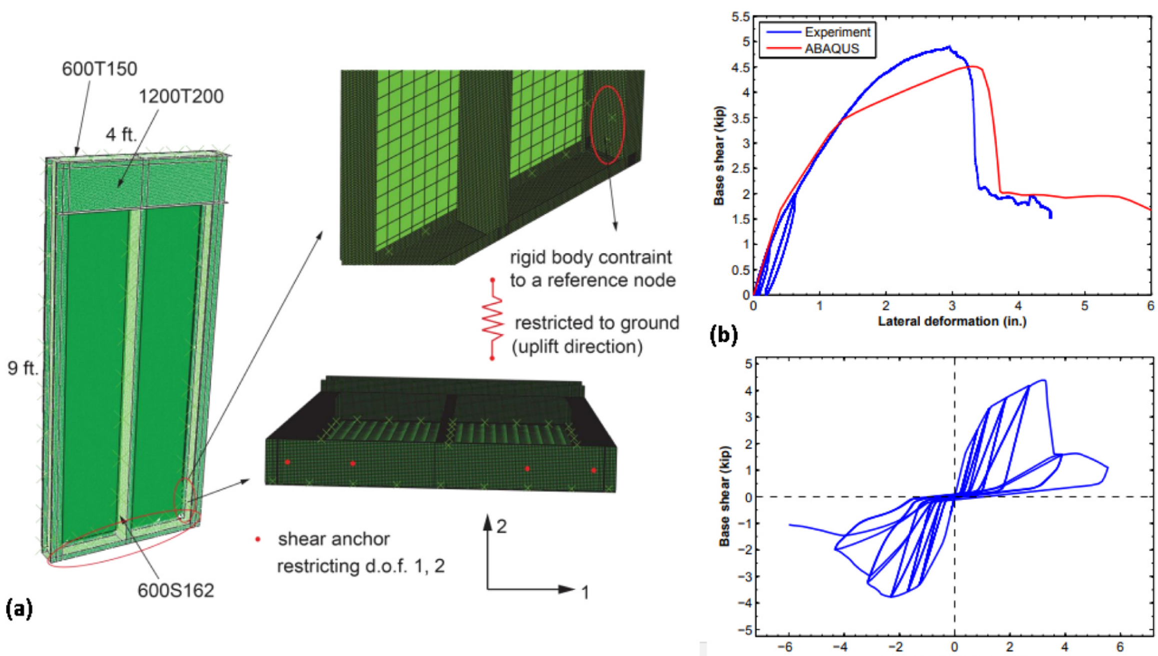


Fig. 34. (a) Micro model of shear wall, (b) comparison of numerical result to experiment, (c) pinching in numerical cyclic response of shear wall model [136].

connection” and “material details, imperfection and element type” is provided in Tables 16–18, respectively. Some modelling details that are not provided by the authors are marked as Not Given (NG) in tables.

4.2. Micro modelling studies by Ansys

Another general purpose FE computer program, being widely used for micro modelling of CFS framed shear wall structures is Ansys [126] software package. The software comprises many special characteristics, which allow non-linearity or secondary effects to be included in the lateral analysis of CFS framed shear walls.

Some of the first numerical analyses by Ansys on CFS framed shear walls were carried out by Gad et al. [44,139,140]. They conducted a detailed investigation into the contribution of plasterboard to the lateral performance of CFS framed residential structures. They implemented both isolated wall panels (with strap and without strap bracing) and walls with full boundary conditions for their purposes. In their experiment and numerical models, the racking load was applied at the bottom, while the top of the house was restrained horizontally only. The equivalent models for strap bracing system as well as the numerical

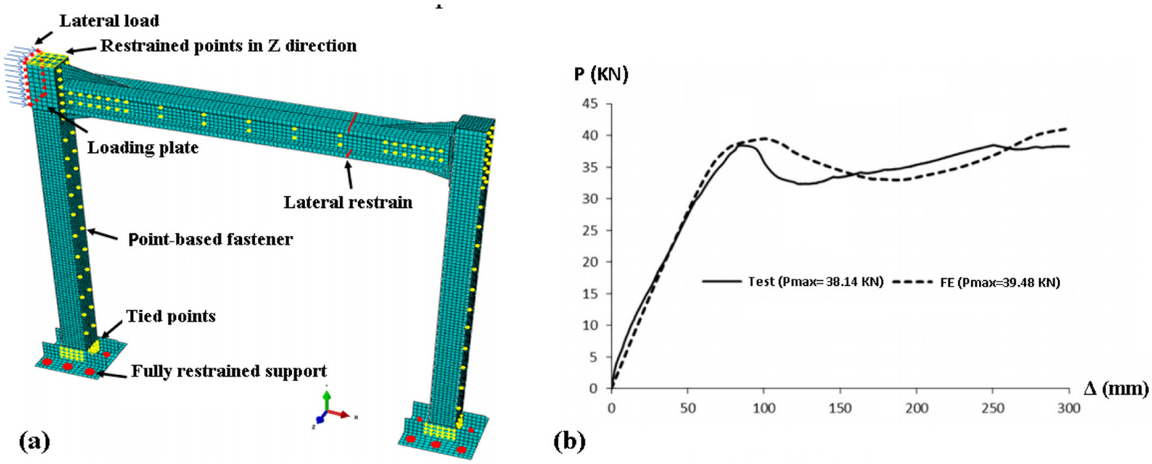


Fig. 35. (a) Typical FE model of the tested CFS moment-resisting frame, (b) lateral load versus lateral displacement [138].

Table 16
A summary of the micro modelling studies by Abaqus, based on boundary conditions.

Author, year, reference	Fixity and loading
Telue and Mahendran, 2004, [128]	Rigid plate for modelling of top track (the track was free to rotate about the global X, Y and Z)
Attari et al., 2016, [129]	Three selected regions on top track for applying load Two selected regions at two sides of the bottom track for modelling of hold-down bolts
Niari et al., 2015, [130]	The hole top track nodes were used for applying load Displacements of bottom track nodes in position of bolts were restrained
Borzoo et al., 2016, [131]	Rigid plate for modelling of top track All parts of the bottom plate in the six degree of freedom were constrained
Hatami et al., 2017, [132]	MPC constraint for top track modelling and for bolts connected to the ground Tie constraint at the point where hold downs are screwed to the stud
Dai, 2012, 2013, [133,134]	Load was applied to the top track via seven points The bottom track was pinned to the ground through 27 points to model the hold-down blots
Yu et al., 2018, [73]	All nodes on web of the bottom track as well as the bottom edges of the studs were constrained in all three directions. For simulation of lateral supports, two lines of nodes on the web of top track were restricted against out-of-plane movements The vertical displacement of all the nodes at the hold-down area of each chord stud was restrained For applying load, all nodes on web of top track were coupled to a reference point located on the edge of the top track
Ngo, 2014, [135]	Rigid plate for modelling of top track
Bian et al., 2015, [102]	Fixing the nodes at the bolt locations (anchor bolts)
Ding, 2015, [136]	Hold down is connected to the ground via a bi-linear spring
Henriques et al., 2017, [137]	Panel anchorage is assumed as rigid Load was applied at the top of the frame (with no rigid assumption)
Mojtabaei et al., 2018, [138]	Rigid plate for modelling of top track The base angles at the place of the bolts were fully constrained by using "Tie" constraint

Table 17
A summary of the micro modelling studies by Abaqus, based on connections and interactions.

Author, year, reference	Connection (Sheathing to frame and frame to frame)	Interaction (Sheathing to frame Frame to frame)
Telue and Mahendran, 2004, [128]	Beam elements with two nodes and six active degrees of freedom for screw connection (is not a perfect pin and is partially restrained)	Smooth interaction with zero friction for sheathing and frame interaction
Attari et al., 2016, [129]	Fastener with Cartesian type and elastic behaviour for screw connection	Surface to surface contact- separation is allowed (for Sheathing to frame interaction)
Niari et al., 2015, [130]	Fastener with Cartesian type with elastic-plastic behaviour for screw connection	Surface to surface contact- for both sheathing and frame interactions
Borzoo et al., 2016, [131]	Nonlinear fasteners with Cartesian type for screw connection	Surface-to-surface contact with a friction factor of 0.2 for all interactions
Hatami et al., 2017, [132]	Elastic spring elements with Cartesian type (translational and rotational links) for screw connection	Surface-to-surface contact with a friction factor of 0.3 for all interactions
Dai, 2012, 2013, [133,134]	3 non-linear spring element for screw connection (one for tension action and the other two for the shear actions)	Surface-to-surface contact with a finite sliding option- friction factor of 0.4 for all interactions
Yu et al., 2018, [73]	Spring2 element was used for modelling sheathing-to-frame and sheathing-to-sheathing screws (3 spring elements for each screw, one withdrawal spring and two shear springs) Tie constraints were used for stud-to-stud and stud-to-track connection	Surface-to-surface contact was sued between the frame and the sheathing (frictionless tangent behaviour and hard-contact normal behaviour were used)
Ngo, 2014, [135]	MPC type PIN for steel-to-steel connections	NG
Bian et al., 2015, [102]	Nonlinear springs for sheathing to frame connection	
Ding, 2015, [136]		
Henriques et al., 2017, [137]	Fully rigid (only in the screw position) for screw connections	
Mojtabaei et al., 2018, [138]	Point-based fastener with beam connector for screw connection	

Table 18
A summary of the micro modelling studies by Abaqus, based on material detail, imperfection and elements.

Author, year, reference	True stress strain	Residual stress	Imperfection	Abaqus element		
				S4R	S4R5	S4
Telue and Mahendran, 2004, [128]		✓	✓		✓	
Attari et al., 2016, [129]			✓	✓		
Niari et al., 2015, [130]	✓		✓	✓		
Borzoo et al., 2016, [131]	✓					
Hatami et al., 2017, [132]			✓	✓		
Dai, 2012, 2013, [133,134]						✓
Yu et al., 2018, [73]				✓		
Ngo, 2014, [135]				✓		
Bian et al., 2015, [102]				✓		
Ding, 2015, [136]				✓		
Henriques et al., 2017, [137]				✓		
Mojtabaei et al., 2018, [138]	✓		✓	✓		

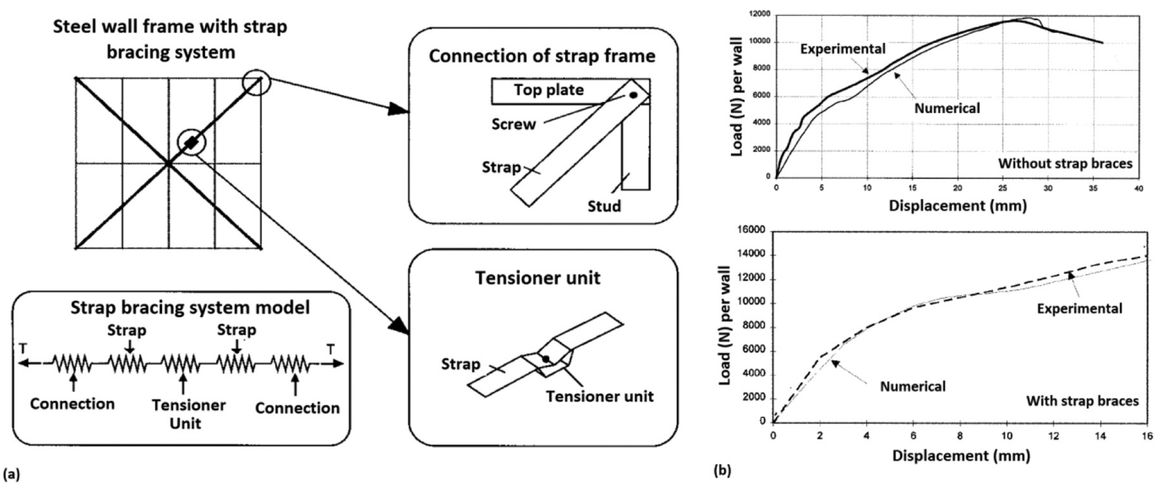


Fig. 36. (a) Equivalent model for strap bracing system, (b) comparison between experimental and numerical results [139].

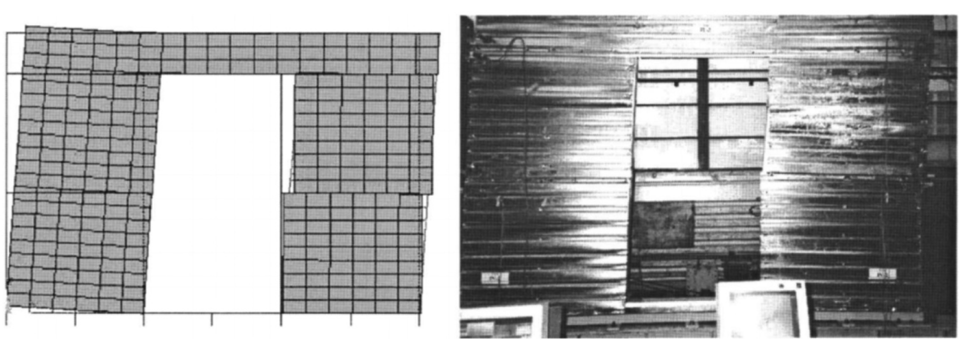


Fig. 37. Comparison of deformation pattern in micro model and experiment [141].

and experimental load-deflection curves for both walls with and without bracing system are shown in Fig. 36. They concluded that the numerical model was able to accurately predict the ultimate load capacity and the deflected shape of frame.

In a micro modelling attempt by Fulop and Dubina [141] to capture the behaviour of shear wall panel with corrugated steel sheets, the corrugated sheet was simulated as an equivalent orthotropic plate (SHELL43). The aim was to consider the basic different mechanical characteristics of the corrugated sheet in two principal directions and the distortion of the corrugated sheet when loaded in shear. As it is depicted in Figs. 37 and 38, a good agreement with experiment in terms of deformation pattern and nonlinear behaviour for large displacements is observed, respectively.

Xuhong et al. [142] employed Ansys to study the shear resistance of CFS stud walls in low-rise residential structures. Based on the verified numerical analyses on Gypsum and OSB sheathed walls, a set of parameter analyses were conducted to study the influence of the steel strength, stud spacing, stud height, and screw spacing on the shear resistance of walls. A numerical study on the lateral performance of shear wall panels sheathed with different materials such as OSB, Canadian softwood plywood, Douglas-fir plywood and gypsum wall board was conducted by Hatami et al. [143]. Using the validated model, they built up a parametric study to examine strength, drift and seismic performance of the shear wall panels. A series of comprehensive nonlinear numerical analyses by Ansys package were carried out by Zeynalian and Ronagh [144,145] to evaluate and optimise the seismic

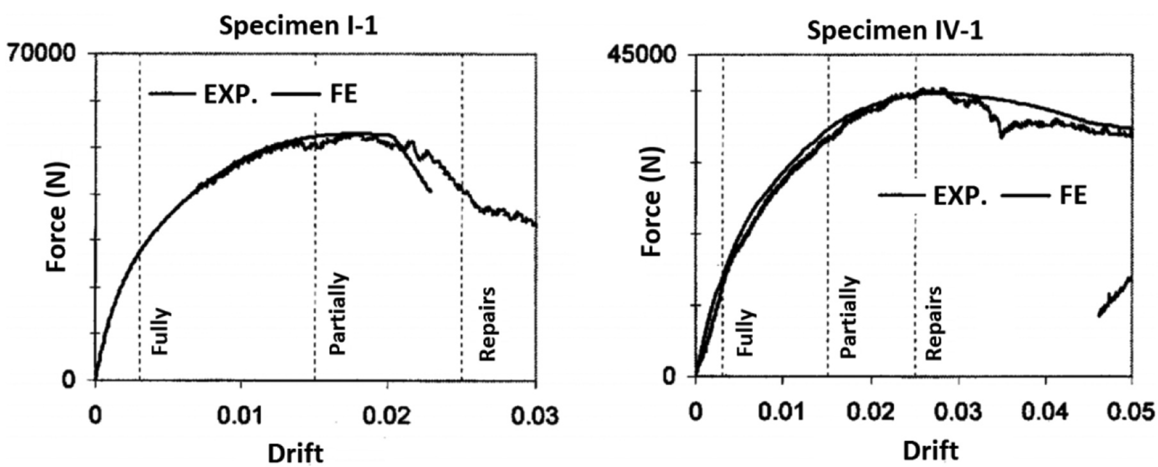


Fig. 38. Comparison of experimental and numerical results [141].

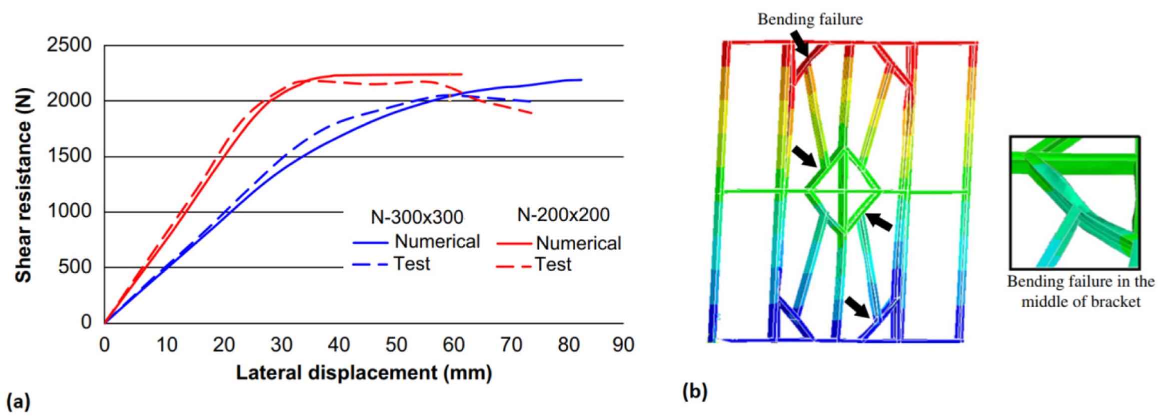


Fig. 39. (a) Experimental and numerical load-displacement curves, (b) failure mode for specimen N-400 [144].

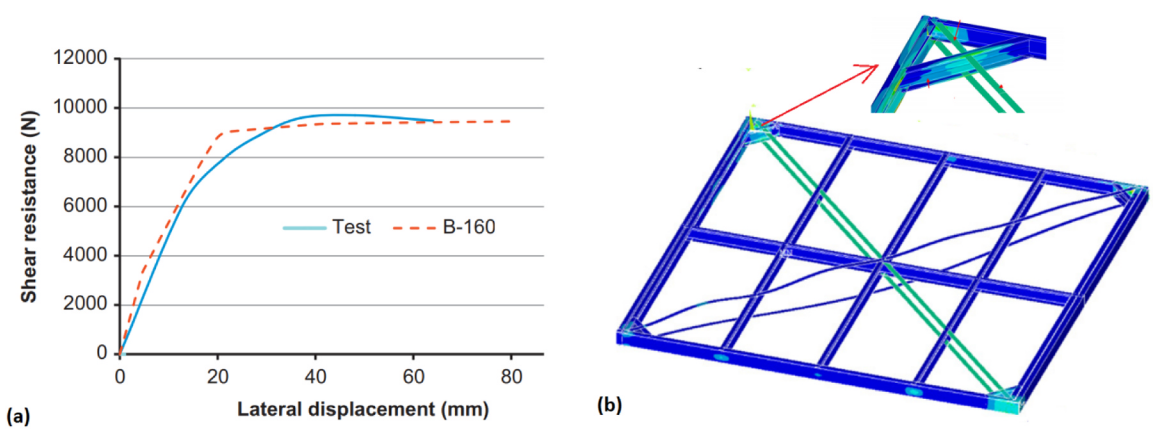


Fig. 40. (a) Experimental and numerical load-displacement curves, (b) deformed X-braced frame [145].

characteristics of knee-braced and strap-braced CFS shear wall panels. Different structural features including: material nonlinearity, geometric imperfection, residual stresses and perforations were taken into consideration in that study. Good agreement between numerical results and experimental data was achieved in their numerical simulations, where the numerical method can be used to predict the ultimate capacity of knee-braced and strap brace CFS shear panels. After validation of the numerical method, they analysed various CFS walls and provided the response modification factor for each panel. Numerical results of knee-braced and strap-braced walls are presented in Figs. 39 and 40 respectively.

In another study, Zeynalian [146,147] carried out a non-linear numerical analysis in order to evaluate the seismic behaviour of steel sheathed CFS walls. To take the fasteners' failure modes into account, Zeynalian modelled the screws connections using COMBIN39 element, which are nonlinear spring elements available in the Ansys package [126]. This element is a unidirectional element with nonlinear generalized force-deflection capability that can be used in a variety of analyses. They concluded that the effects of residual stresses and section perforations are negligible. In another study, Abu-Hamd et al. [148] performed nonlinear verification analysis for CFS-braced shear walls and implemented parametric study using the nonlinear FE model. They

Table 19
A summary of the micro modelling studies by Ansys, based on boundary conditions.

Author, year, reference	Fixity and loading
Gad 1997a,b, 1999, [44,139,140]	Pin connections were used for hold down modelling
Fulop and Dubina, 2006, [141]	Bars of the skeleton were modelled as elastic beam elements (BEAM4) (it is assumed that elements were not highly deformed in the post elastic range)
Xuhong et al., 2006, [142]	The nodes of top track were coupled for applying load Displacements along the X, Y and Z-directions and rotations along Y and Z-directions of bottom track were restrained
Hatami et al., 2014, [143]	The nodes of top track were coupled to one node for applying load
Zeynalian and Ronagh, 2012, 2011, [144,145]	NG
Zeynalian, 2015, 2017, [146,147]	Coupling command for relevant nodes at bottom track
Abu-Hamd et al., 2018, [148]	Reference nodes defining the hold-down elements are connected to nodes on the ground in the vertical direction via a bilinear spring element type COMBIN39

Table 20
A summary of the micro modelling studies by Ansys, based on connections and interactions.

Author, year, reference	Connections (Sheathing to frame and frame to frame)	Interactions (Sheathing to frame Frame to frame)
Gad 1997a,b, 1999, [44,139,140]	Pinned connection was used for tab-in-slot connection of the frame Four springs with Non-linear behaviour for modelling of screw connection	NG
Fulop and Dubina, 2006, [141]	Connections, both between the skeleton and the sheathing and the seam connections, were modelled using COMBIN39 elements	NG
Xuhong et al., 2006, [142]	Coupling method for modelling of screw connections (with free rotations but no displacement)	Without considering the slip between sheathing and steel members
Hatami et al., 2014, [143]	Coupling for modelling of screw connections (sheathing to frame)	NG
Zeynalian and Ronagh, 2012, 2011, [144,145]	Coupling technique for modelling of rivet connections	
Zeynalian, 2015, 2017, [146,147]	Coupling technique for modelling of rivet connections Screws connections were modelled using COMBIN39 (Consider the fasteners failure)	
Abu-Hamd et al., 2018, [148]	Coupling set for shear nonlinear failure in-plane mode of the screws using two nonlinear spring elements COMBIN39	Contact elements surface to surface (TARGE170 and CONTA174) for connections between the bracing and the gusset plate

Table 21
A summary of the micro modelling studies by Ansys, based on materials, imperfections and elements.

Author, year, reference	True stress strain	Residual stress	Imperfection	Ansys element		
				shell181	Shell43	Shell93
Gad 1997a,b, 1999, [44,139,140]						✓
Fulop and Dubina, 2006, [141]					✓	
Xuhong et al., 2006, [142]				✓		
Hatami et al., 2014, [143]			✓	✓		
Zeynalian and Ronagh, 2012, 2011, [144,145]		✓	✓	✓		
Zeynalian, 2015, 2017, [146,147]		✓	✓	✓		
Abu-Hamd et al., 2018, [148]			✓	✓		

also verified and thus investigated the response reduction factor R.

The micro modelling studies on the lateral behaviour of CFS framed shear wall structures using Ansys computer package, in the literature, was reviewed in this section. A summary of the modelling techniques classified based on the "boundary condition", "interaction and connection" and "material details, imperfection and element type" is represented in Tables 19–21, respectively. Some modelling details that are not provided by the authors are marked as Not Given (NG) in tables.

4.3. Micro modelling studies by Sap2000

One of the widely used engineering software is Sap2000 [127], which is ideal for both micro and macro modelling of CFS components. Although micro modelling by this software is not as accurate as Abaqus and Ansys, it is broadly used in practice and research for the FE modelling and simulation of framed structures. Yet quite few research studies have employed this software for the micro analysis of CFS shear walls.

Some simplifications need to be implemented in micro modelling strategy when analysis of a mid-rise CFS building by micro methods. In this context, Martinez and Xub [149,150] simplified the conventional micro method by suggesting that the individual modelling of the studs and sheathing plates in a building is not necessary. Instead, the walls were transformed into a sixteen-node shell element with equivalent properties for modelling complete panels. Fig. 41 displays both conventional and simplified micro modelling of a CFS shear wall, as well as the developed model for a three-storey building by the proposed simplified micro method in that study. Results of the simplified micro method, compared with the experimental data, are shown in Fig. 42. In linear analysis of the single shear wall, it was explained that the results obtained from simplified micro method for isolated wall of various lengths are in good agreement with those captured from conventional micro method. In the building model, the lateral displacements obtained by conventional micro method were also in good agreement with those captured by simplified micro method. The internal forces were overestimated in simplified micro method though. They suggested that the proposed simplified micro method can be used to provide global performance of the structure such as lateral deformation of the building and lateral forces in walls. Due to the differences in the axial forces of the studs between the results from simplified and conventional micro

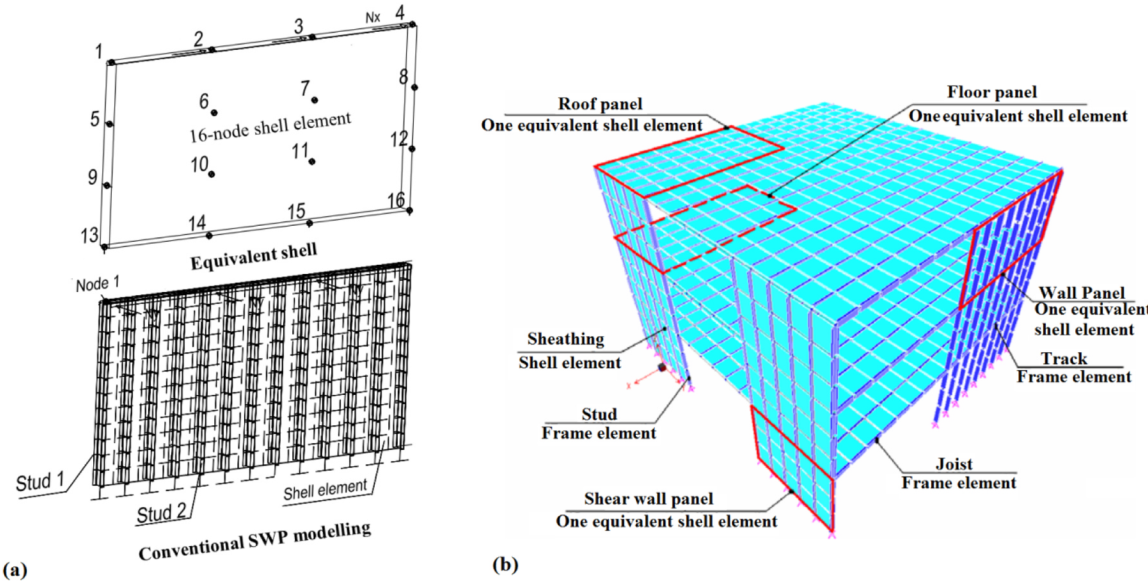


Fig. 41. (a) Conventional and simplified micro modelling of CFS wall, (b) modelling of a CFS building using simplified micro method [149].

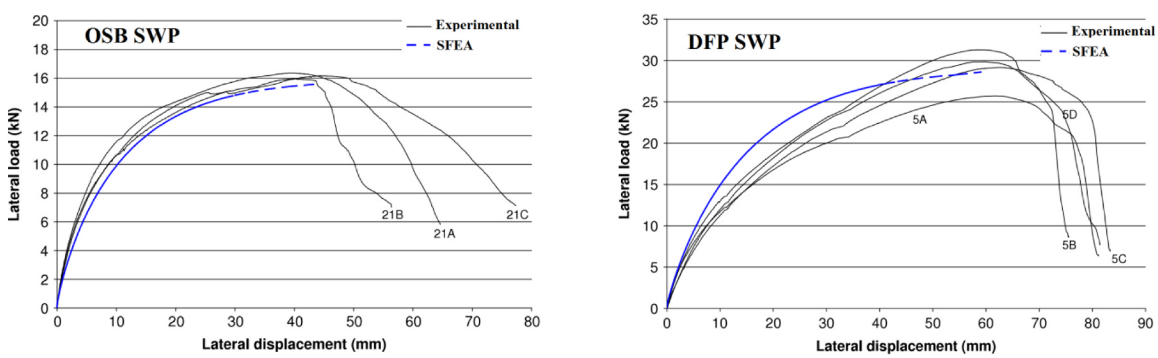


Fig. 42. Numerical vs. experimental curves of CFS walls [149].

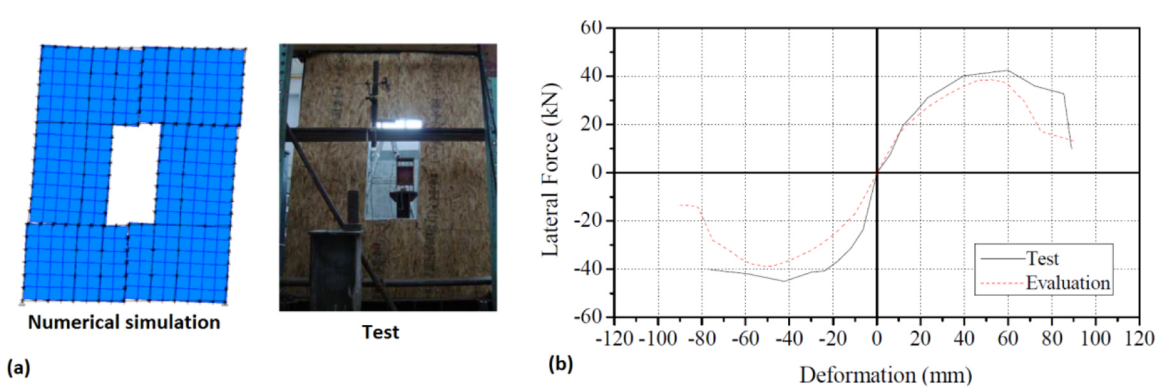


Fig. 43. (a) Deformation comparison of specimen SW6, (b) load vs. deformation curve comparison of specimen SW11 [143].

method, they recommended not to use the axial forces obtained from simplified micro method for design of the steel stud of wall.

Li et al. [143] performed a refined numerical simulation of CFS shear walls based on modified exponential Foschi skeleton [151] to simulate the behaviour of stud-sheathing screw connections in shear loading. The modelling was carried out on walls sheathed with OSB and gypsum boards. Studs and sheathings' connections were modelled using two-freedom spring elements in order to capture deformation along and perpendicular to the loading directions. Fig. 43 illustrates how the numerical results, including deformed shape as well as load vs. deformation curves agree with the tests. It indicates that the behaviour of shear walls can be precisely assessed through the numerical simulation technique proposed in that study.

Karabulut and Soyoz [152] provided numerical models by Sap2000 for the study of CFS shear panels to be used for 3D structural analysis and seismic performance investigation. They verified the modelling approach by simulating a CFS bare frame with sheathing board. Their geometry model of single wall and comparison of FE and experimental results are shown in Fig. 44a and Fig. 44b respectively. Since a relatively good agreement between the numerical and experimental results

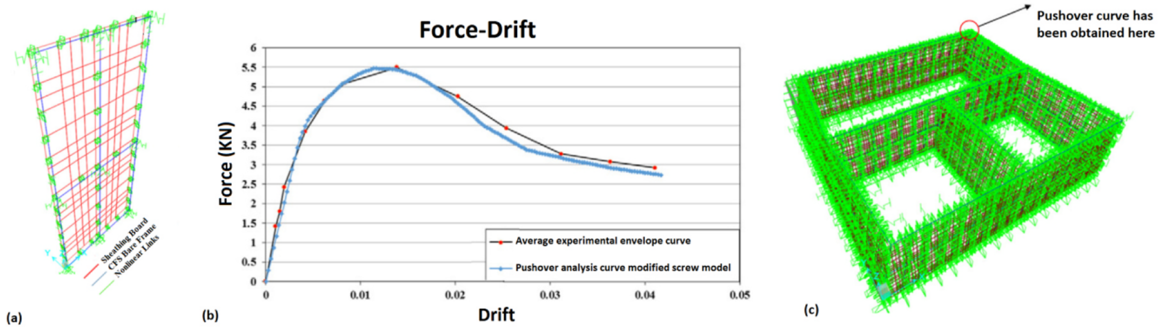


Fig. 44. (a) Numerical model of the CFS shear panel, (b) comparison between numerical and experimental results of type 6 CFS shear panel, (c) one-storey residential building [152].

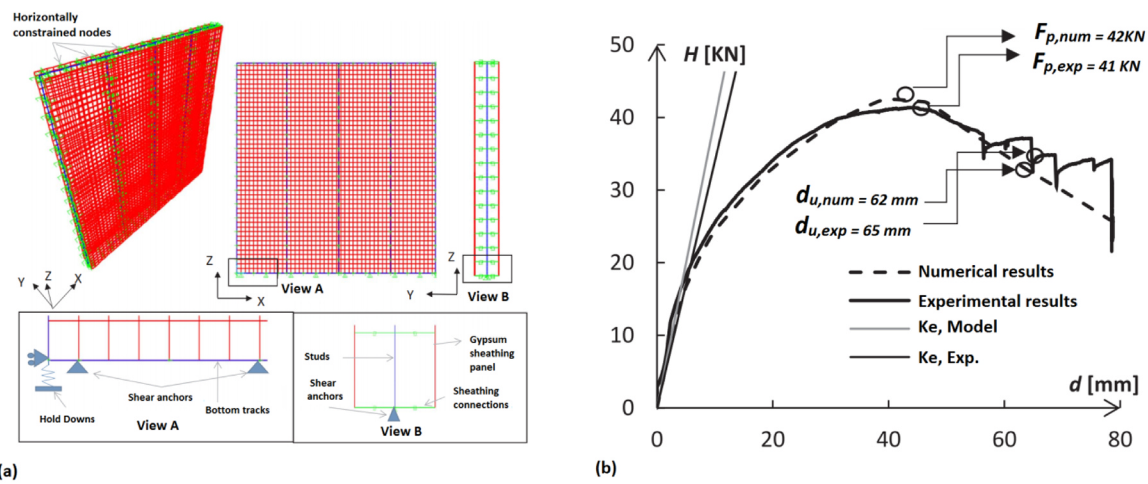


Fig. 45. (a) Schematisation of micro model developed for CFS sheathed braced shear walls, (b) comparison of experimental and numerical results [76].

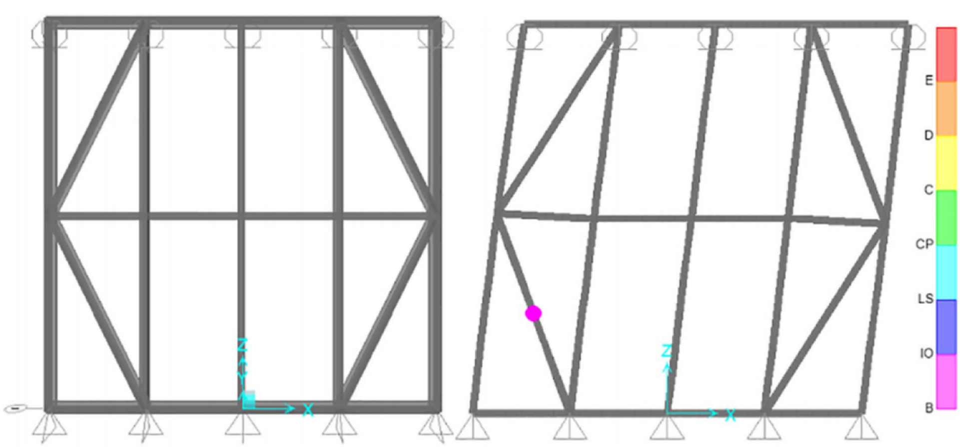


Fig. 46. Nonlinear static analysis of the K3 model before and after formation of the first hinge in the brace element [153].

was captured, they developed the numerical model to be implemented in a one-story residential building. The building's model is presented in Fig. 44c. Their study tried to assure the capability of CFS structures to possess a sufficient performance in seismic prone regions such as Turkey.

Recently a new numerical model was proposed in Sap2000 by Fiorino et al. [76] for CFS shear walls with capability of capturing their nonlinear behaviour. It was indicated that the numerical models are in good agreement with experimental observations in terms of deformation mechanism. They implied that the models developed in their study can be useful for a practicing engineer to calculate strength and stiffness of single shear wall. Fig. 45 shows schematisation of their numerical model developed in Sap2000 for CFS sheathed braced shear walls as well as comparison of the short wall numerical models (WS_2400_M) with its monotonic test response.

In another study, Pourabdollah et al. [153] conducted a brief numerical simulation to predict the shear strength of K-braced CFS shear panels for practical simple engineering design. Having defined the plastic hinges at the mid-length of the brace elements, nonlinear static analysis was performed for a target displacement. As Fig. 46 depicts, the first plastic hinge occurred at the brace element under compression around a drift of 2.5%.

Table 22
A summary of the micro modelling studies by Sap2000, based on boundary condition.

Author, year, reference	Fixity and loading
Martinez and Xub, 2007, 2011, [149,150]	Bottom track was not included in the model because the bottom nodes are fixed sheathing is modelled using square four-node shell elements Load was applied at the top four nodes in the x and y directions
Li et al., 2014, [143]	Top and bottom girders were considered as the rigid members by means of increasing their elastic modulus.
Karabulut and Soyoz, 2017, [152]	NG
Fiorino et al., 2018, [76]	Rigid plate for top track modelling A simple support, and a vertical linear springs for modelling of hold-downs Pinned restraints for modelling of shear anchors
Pourabdollah et al., 2017, [153]	Fixity is applied under each stud location

Table 23
A summary of the micro modelling studies by Sap2000, based on connection and interaction.

Author, year, reference	Connection (Sheathing to frame and frame to frame)
Martinez and Xub, 2007, 2011, [149,150]	NG
Li et al., 2014, [143]	Connections among the framing members were modelled as hinges Connections between the studs and sheathings were modelled using two-freedom spring elements
Karabulut and Soyoz, 2017, [152]	Nonlinear link elements for screws (moment is released for the connections)
Fiorino et al., 2018, [76]	Hinge technique for modelling of frame connection Two nonlinear link element with a multi-linear backbone curve for modelling of sheathing connection
Pourabdollah et al., 2017, [153]	Plastic hinges were used at the mid-length of the brace elements

As stated above, because micro modelling simulation of CFS framed structures in Sap2000 is not as accurate as modelling by Abaqus and Ansys, a lower number of micro modelling studies by this software exists in the literature. In addition, there is not clear detailed information about the modelling strategies in Sap2000, with regard to CFS framed walls under lateral loads. A brief summary of the modelling techniques in terms of "boundary condition" and "interaction and connection" using Sap2000 is given in Tables 22 and 23 respectively.

5. Comparison and discussion

Based on the research studies reviewed in Sections 3 and 4, it can be concluded that the analysis with well-developed macro-models can provide the same accurate results as micro-models. Yet, the findings indicate that the accurate representation of all structural elements by micro method can be relatively too complicated, and result in disproportionate computational cost, when modelling the entire structure. A micro model of a CFS frame that includes all aspects of the problem needs the consideration of many geometric details and contact relations between several parts of the frame, as indicated in Fig. 29. Knowing that, even for a simple CFS panel containing a great number of fasteners, analysing the entire structure with all its complexity can lead to unreasonably long computational time. This was the motivation for many of the proposed macro models.

The aim of many research studies of developing strong macro models for the analysis of CFS frames has been to reduce the difficulty of the geometry and the number of contact relations, so that the analysis can be finalized in a reasonable time, without significantly compromising the accuracy. This is even more evident, when it comes to large scale structures, such as multi-storey buildings made of CFS elements [47,48,53,55–60,62,64–73,77,78,80,81,83,86–90,95–99].

Furthermore, micro-models have been mainly used only for detailed analysis of local responses of small CFS framed structures such as wall panels [44,73,76,102,128–150,152,153]. Some micro models however, have been used as the input to determine individual element and node responses. Conversely, in some cases the output from a macro model has been used to determine the properties of a simple global model, and then has been employed to derive global responses (e.g. storey drift), through repeated analyses.

The proposed macro methods, on the other hand, have shown that are not able to account for the local failures (detachment of sheathing to frame, screw pull out and failure of stud, track and sheathing) and the effect of different buckling modes on the entire structures behaviour; thus, not applicable for problems in which buckling analysis is the case. In most macro models, it has been also assumed that the hold-down anchors and screw connections have been properly designed to resist the entire forces in the wall panels. Therefore, hold-down anchor and screw failures are not accounted for in the determination of the walls' lateral strength. Quite the contrary, the local failures and buckling modes, as well as hold-down and screw failure mechanisms can be captured from the micro methods.

In both micro and macro methods, it is mainly suggested that the rigid panel assumption may not be appropriate for shear walls sheathed with steel sheathing, which undergo substantial deformation within the panel and smaller deformations around the fasteners. In macro modelling methods, the CFS framing members (stud, ledger or track) are mostly modelled by elements assumed to be rigid cross-sections and do not allow for localized plate flexibility in the CFS framing. In most proposed macro modelling methods, extracting the graphical results, such as stress and strain distribution and deformation, is not also possible.

In dynamic and cyclic analysis of CFS shear walls by micro method, some issues may occur due to unknown problems of numerical algorithms for time integration. Dynamic analysis needs a large number of relatively small finite and contact elements, i.e. a greater number of uncertainties in the system. In addition, pinching phenomenon usually occurs in real CFS structures, which need to be accounted in numerical modelling. Technically, micro modelling methods have some limitations in capturing pinching behaviour of fasteners. This phenomenon therefore, can be modelled by defining a subroutine to incorporate a physical gap between screws and holes' bearing faces in the software that may need considerable effort. Hence, due to the lack of a proper models for simulation of pinching behaviour in screws and walls, almost all micro modelling studies (except Ding [136]) have been carried out under monotonic loading and static analysis. The proposed macro modelling methods on contrary can simply capture the pinching behaviour by defining the hysteresis models during simulation, so that all monotonic, cyclic and seismic actions and responses can be conveniently simulated.

There have been some discrepancies between numerical results and

Table 24
Reason for discrepancy of numerical and experimental results for macro modelling method.

	Method	Author, year, reference	Reason for discrepancy of numerical and experimental results
Macro modelling	**Equivalent brace**	Shamim and Rogers, 2012, [56,60]	– Sheathing can be detached in walls during experiment; however, this is not happening in numerical method. – Employing smaller damping value in numerical method can be resulted in an increase in the differences between numerical and experimental results.
		Leng et al., 2017, [65]	– Damping value in numerical method is held constant at 5%, while in the tests the damping is increased with construction phase. – Simpler simulation of vertical load path at the floors in the simulation compared to the actual building. – Other parameters such as: the conservative estimation of connection stiffness as fixed, the accuracy of stiffness estimate of gravity and interior wall sheathing and semi-rigid diaphragms as subpanels and the approximation of built-up members as isolated ones.
		Leng et al., 2016, [66]	– Modelling of the wall-ledger-diaphragm interactions (line elements may fail to fully capture the stiffening effect of thick ledgers and joists in boxing and stiffening this top of the shear and gravity walls). – Inter-story stiffness of the chord studs are is not accurate. – Conservative assumption of zero tensile strength of shear anchors along the gravity walls limiting lateral strength and coupling ability
		Fiorino et al., 2017, [76]	– Underestimation of post-peak response is due to post-peak shape of sheathing connections backbone envelope, which is characterized by a higher slope respect to those showed by walls
		Macillo et al., 2018, [79]	– Strain hardening of the material is not taken into account for the theoretical backbone curve utilized in numerical method resulting in the under-prediction of strength and energy dissipation
		Boudreault, 2005, [47]	– The Stewart Model is unable to capture failure of the wall and doesn't offer strength degradation
		Foutch and Lee, 2010, [53]	– Straps in the numerical method are simulated to be tightly attached to the columns, while attachment of the straps in test is loose – Base of the shake-table is flexible in test due to the oil columns and vertical actuators, but the base is assumed to be fixed in the numerical model
		Foutch et al., 2007, [55]	– Complete control of the shake-table during experiment was not possible – Since the effective lengths of the straps are not identical (as welded in place), little slack of some straps occurs which is not happening in numerical method – Deformation and separation of some anchors from the slab during the test which is not happening in numerical method
	Equivalent spring	Kechidi and Bourahla, 2016, [94]	– Sheathing can be detached in walls during experiment, while this is not happening in numerical method – The model does not dissipate energy below predicted value, but energy dissipation is exhibited in experimental test at displacement level below the predicted value.
		Jiang and Ye, 2018, [99]	– Overestimation in lateral stiffness is due to rigid connection assumption at column base in numerical method – The natural frequency of the numerical model is higher than the test – Underestimation in energy dissipation of the structure is due to the Pingching04 hysteretic model. Because the unloading stiffness is lower than the loading stiffness, but the unloading stiffness of the test shear wall is larger than the loading stiffness. – Pin connection simulation for connections between the CFS beams and CFRST in numerical model prevents to capture the energy dissipation of these connections
	Fastener based	Bian et al., 2014, [100]	– Additional flexibility and redistribution in real wall which is not included in the numerical model. – Degrading branch in the Pinching04 "fastener" model is possibly too severe – Rigid sheathing assumption doesn't create a favourable load distribution to the fasteners
		Buonopane et al., 2014, [104]	– Fully pinned modelling of shear anchors which is not similar to test – The per cycle error between energy dissipated in the numerical model and tested specimens – Numerical models may fail is a smaller magnitude peak displacement cycle than the physical tests
		Padilla-Llano et al., 2015, [107]	– Discrepancies in post-peak monotonic response is due to the use of the contact springs along the bottom track

experimental data reported for both micro and macro methods. The reasons for such discrepancies are summarised in Tables 24 and 25 respectively

Considering the findings of research studies reviewed in this paper, the authors' reflections on the present and future of the CFS shear walls modelling are as follows:

5.1. For micro models

- In micro models, the lack of simulating the small friction between CFS elements has been reported as one of the reasons for discrepancy between experimental and numerical results (Table 25), while there is also no generic measure proposed for estimating the friction coefficient between CFS structures elements. In addition, some researchers have also employed different friction coefficients in their simulation (Tables 17 and 20), which needs further discussions on how the coefficient is calculated.
- Static analysis is an effective and accurate solver for capturing the behaviour of CFS shear walls under monotonic loading. Nevertheless, static analysis is either numerically expensive or sometimes fails to converge due to unstable collapse or post-buckling. To avoid these issues, other analysis options such as dynamic and riks method can be also used for such simulation. While the type of solver is rarely mentioned in the previous researches, the effects of using alternative and applicable solver options (dynamic and riks) should be also evaluated.
- Although some good numerical results have been captured by different models, there is not yet a general modelling approach for simulation of CFS shear walls (Tables 16–23). As an example, considering initial imperfection, residual stress and true material properties (e.g. stress-strain relation), as well as the type of elements used in modelling may or may not influence the numerical results (Tables 18 and 21). The effects of these parameters on numerical results need to be exactly evaluated in the future research.
- A major drawback of micro modelling method is the problem with simulating cyclic behaviour of screws with the ability of capturing pinching behaviour. Although Ding [136] provided a UEL code in Abaqus for capturing the hysteresis behaviour of CFS shear walls,

Table 25
Reason for discrepancy of numerical and experimental results for micro modelling method.

	Method	Author, year, reference	Reason for discrepancy of numerical and experimental results
Micro modelling	**Shell element computer programs**	Attari et al., 2016, [129]	– Screw pull out phenomenon and imperfections of tested specimens are not modelled in numerical method which cause some discrepancies
		Mojtabaei et al., 2018, [138]	– Small friction in the lubricated interface of the beam and bracing elements in test which is not happening in numerical model – Fixed bearing connections in the FE models can provide additional strength, while no additional strength is observed in test.
		Esmaeili et al., 2015, [130]	– It is the nature of the numerical method to have higher stiffness than the real experimental specimen.
		Telue and Mahendran, 2004, [128]	– Ultimate strength of studs in numerical method may be lower than experimental test which may be due to considering geometric imperfections and residual stresses in numerical model
		Ngo, 2014, [135]	– Assumption of rigid or semi-rigid diaphragm for sheathing which is not same as experimental test – The error in test results itself when only one specimen is tested for each shear wall configuration – Modelling hold-downs as springs does not capture moment of the couple consisting of axial force in chord studs and reaction force on the hold-down rod from the foundation. However, in the tests, the anchor rod connecting hold-down to the foundation is slightly offset from the line along chord studs web.
		Ding, 2015, [136]	– The stiffness of the members is underestimated
		Hatami et al., 2017, [132]	– The tested walls also have some differences in results (experimental errors) – Although the failure of any screw connection has a major impact on convergence phenomenon, it has no significant impact on displacement. But, this is different in experiment.
		Hatami et al., 2014, [143]	– Some factors such as accuracy of measuring tools and experimental errors including, imperfection of shear wall panels, cracking of wood and gypsum panels, etc. – It seems difficult to construct a model which is completely compatible with several experimental results.
		Martinez and Xu, 2011, [149]	– Even though the sheathing is attached to the studs in test, numerical model considers the shell and frame elements in the same plane, so that the offset of the sheathing from the centerline of the studs is not considered. – The type, size, and number of elements employed to simulate a structure can affect the accuracy of results.
		Bian et al., 2014, [100]	– Application of two separate translational in-plane springs for modelling of steel to sheathing connections in the board plane which is not exactly the same in the test
		Fiorino et al., 2018, [76]	– Higher slope of sheathing connections backbone envelope, in comparison to those showed by tested walls.
		Emad Gad, 1997, [44]	– Using higher shear modulus for plasterboard modelling than real experimental shear modulus
		Yu et al., 2018, [73]	– Small sliding during experiment, which is not considered in numerical method
		Henriques et al., 2017, [137]	– Screw connection modelling which is assumed more rigid in the numerical model than in the experimental test

this issue is still a big concern for micro modelling of these systems.

- Considering a rigid body motion for simulation of sheathing can decrease the computation time and convergence problems. Yet, the accuracy of results can be significantly affected. In addition, despite some researches having utilized rigid body mechanism for their simulation, there is not detailed information available on their assumptions for modelling. Numerical modelling with both rigid body and semi-rigid body assumptions and a comparison can help better understand this matter.
- Combining micro modelling method by some macro modelling assumptions can decrease the analysis time, while maintain the accuracy of capturing good results; nonetheless, it requires substantial research and model development. Implementation of both micro- and macro modelling methods in one numerical simulation, as it was performed by Martinez and Xu [149], can also provide a valuable comparison.
- Modelling of connections, as well as the interaction and fixity between members in shear walls have been modelled in some studies with different techniques (Table 16, Table 17, Table 19, Table 20, Table 22, and Table 23). A comparison between these modelling techniques in terms of computational time and the accuracy of results can better help understand the strength and weaknesses.
- Although capturing the general failure mode of the CFS walls (such as cracking, braking and tearing of sheathing, deformation of studs or tracks, wedge failure and failure of strap [154–156]) is feasible and easy in micro modelling approach, modelling the failure of the screw connections such as pull-through failure, screw rotation, tearing of screw, etc. still requires more effort [157,158]. Some attempts have been made by Selvaraj and Madhavan [157] and Zeynalian [146,147] in order to capture the correct screw failure mode, which can be used for future models.

5.2. For macro models

- While different macro modelling programs have been developed in the literature, there is no specific section or toolbox for CFS modelling. An attempt has been made by Kechidi and Bourahla [94] in order to incorporate cyclic behaviour of CFS shear walls in OpenSees software by developing and implementing CFSWSWP and CFSSSWP materials in this program.
- Pinching4 hysteresis curve is mostly used for modelling of CFS shear walls under cyclic loading [56–60,62–73,76–80,94–107]. A general data-base for Pinching4 parameters based on the previous experiments with different sheathing materials can significantly facilitate the future modelling methods.
- Most of the macro modelling software programs do not provide graphical user interface neither for modelling nor results. Computer program such as OpenSees, which is used by a large number of researchers in this area, requires less complex modelling strategies, better graphical user interfaces and a mechanism to identify the possible errors.

For the behaviour analysis of CFS shear walls, various model reduction techniques have been developed and used. Generally speaking, the implementation of a technique for analysing a system is quite case sensitive, and their performance is quite dependent upon the appropriate choice of an effective micro- or macro-model that most suits the case. Although they work well in each of their purpose, there still remain hurdles such as comprehensive parametric methods that can be used for a variety of cases. In fact, with most proposed method, when parameters of a system change, the entire system is changed; the model is no longer valid; and consequently modelling, reduction, and analysis should be repeated for each case. Future research on the parametric model reduction techniques for CFS walls and panelised systems can pave the way for more efficient applications of these systems in the construction industry.

6. Concluding remarks

The advancement in CFS structures has been the results of a combination of developments in applications and improvements in simulation. The increased intention of using these systems in more complex structures worldwide has placed engineers and researchers under some pressure to find adequate modelling techniques for simulation of these structures under lateral loads. The emphasis of this paper was to summarise and review the major research developments of numerical research related to lateral performance of CFS framed shear wall structures, and study their strengths, limitations and contributing factors, and parameters to their performance. The existing models were categorised under micro- and macro- modelling methods, and a summary of the modelling techniques, hysteresis models and the reason for discrepancy between experimental and numerical results were provided. The work is limited by the scopes of paper, as well as the length of the publication, as there are more details that are worth to be further discussed.

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