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COLD FORMED STEEL SHEAR WALL RACKING ANALYSIS THROUGH A MECHANISTIC APPROACH: CFS-RAMA

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ABSTRACT

Cold-formed steel shear wall panels are an effective lateral load resisting system in cold-formed steel or light gauge constructions. The behavior of these panels is governed by the interaction of the sheathing - frame fasteners and the sheathing itself. Therefore, analysis of these panels for an applied lateral load (monotonic/cyclic) is complex due to the inherent non-linearity that exists in the fastener-sheathing interaction. This paper presents a novel and efficient, fastener based mechanistic approach that can reliably predict the response of cold-formed steel wall panels for an applied monotonic lateral load. The approach is purely mechanistic, alleviating the modelling complexity, computational costs and convergence issues which is generally confronted in finite element models. The computational time savings are in the order of seven when compared to the finite element counterparts. Albeit its simplicity, it gives a good insight into the component level forces such as on studs, tracks and individual fasteners for post-processing and performance-based seismic design at large. The present approach is incorporated in a computational framework - CFS-RAMA. The approach is general and thereby making it easy to analyze a variety of configurations of wall panels with brittle sheathing materials and the results are validated using monotonic racking test data published from literature. The design parameters estimated using EEEP (Equivalent Energy Elastic Plastic) method are also compared against corresponding experimental values and found in good agreement. The method provides a good estimate of the wall panel behavior for a variety of configurations, dimensions and sheathing materials used, making it an effective design tool for practicing engineers.

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1. Introduction

Cold Formed steel (CFS) framed construction is one of the most resilient system in residential, commercial and institutional constructions. The use of cold formed steel as secondary non-load-bearing components like partition walls and curtain walls has been for long time in construction industry. But with the increase in research on these members/systems as primary load-bearing members and thereby inclusion in the codes of practice, it is gaining appreciable response for adoption in construction sector. Cold formed steel sheathed wall panels (CFSSW) form the gravity and lateral load resisting systems in these constructions. They consist of framing, sheathing, fasteners and hold downs. The framing consists of studs which are cold formed steel lipped channel sections arranged vertically and tracks are un-lipped channel sections, which hold the studs together at top and bottom locations. Together they form a framing system. Sheathing serves as skin for the steel framing and also braces the studs in-plane and out-of-plane directions due to being attached through fasteners at discrete locations. Figure.1 shows the schematic layout of a cold formed steel shear wall panel.

The response of CFSSW panels to applied lateral load well into the non-linear range is of importance for designers to elicit proper design guidelines for Performance-Based Seismic Design (PBSD). The CFSSW panels can be broadly classified into 3 types namely, brittle sheathed panels, ductile sheathed (steel sheet) panels and strap braced panels based on the type of sheathing material used and failure modes observed. The design guidelines for ductile sheathed and strap braced wall panels is well established in AISI S213[1], owing to the extensive experimental campaign by [2, 3, 4]. But the design of brittle sheathed wall panels such as those sheathed with Gypsum boards, calcium silicate boards, fiber cement boards, Oriented Strand Boards (OSB) and so on, require special attention due to their highly non-linear material characteristics. The lateral response of these panels depends mainly on the behavior of the screws connecting the frame to sheathing. This fact is corroborated by the extensive experimental research in the past two decades on these panels with different configurations and sheathing materials. There had been significant effort by researchers across the globe to develop models for assessing the response of wall panels based on the response of individual fasteners, assessed experimentally. These models usually employ finite element analysis to capture the behavior of CFSSW panels. Therefore although accurate, they are prone to complex modelling and convergence issues by practicing engineers.

There had also been some mechanistic and empirical approaches developed to predict the ultimate load and displacement or peak load and displacement of CFSSW panels subjected to monotonic lateral load. However, these are limited in their scope of application due to the inherent assumptions and empiricism. Moreover they do not predict the entire load deformation history. But, the entire non-linear load deformation is necessary to calibrate equivalent single degree of freedom models to incorporate in full scale building models for response prediction of CFS structures.

This paper presents a simplified, robust and yet reliable mechanistic approach which utilizes the data from the fastener shear tests and predicts the entire non-linear load deformation history of the CFSSW panel. This is a step taken towards enabling analysis of different configurations of wall panels alleviating the necessity for full scale wall panel tests which are in themselves costly and difficult to perform. Modelling effort made by past researchers is reviewed and proposed approach is discussed, followed by implementation of the proposed approach in predicting response of four different configurations of CFSSW with different rigid panels from literature. Also the results and post-processing capabilities are presented, followed by conclusions and future work.

1.1. Background research on response prediction of CFSSW panels - analytical and numerical methods

To a large extent the behavior of CFSSW panels closely resembles the behavior of sheathed wood shear wall panels (WSW). It has been a legacy in wood-framed structures, to analyze the behavior of wood shear walls using the screw load-displacement data obtained by simple screw tests. Early efforts by Tuomi and McCutcheon [5], Easley et al. [6], Mc-Cutcheon [7] were to derive simple analytical formulas to predict load-displacement history of WSW under monotonic lateral loads, based on the screw tests and using an energy approach. Although these closed form equations captured the load-deformation behavior accurately up to a moderate load levels, the simplified assumptions in behavior of wall panel and in approximating the screw load deformation data to a function, prevented them to capture the behavior fully into the non-linear range. Itani et.al [8], Dolan and Foschi [9], White and Dolan [10] adopted finite element models to analyze the behavior of these WSW panels. In this regard two computer programs were developed by the researchers namely, SHWALL and WALSEIZ, used for assessing lateral behavior of WSW panel. The models model the frame with beam element, sheathing with 4 node plate element and sheathing to framing connector as non-linear spring. Although, these models are comprehensive and accurate, it is often difficult for a practicing engineer to adopt such methodologies in design practice. Moreover, they are computationally too expensive while modelling whole building for non-linear dynamic analysis [11]. Gupta and Kuo [12] on the other hand followed the similar methodology as [5] and [7], but their aim was not to arrive at simple closed form solutions, rather to develop an analytical procedure based on strain energy approach to arrive at equilibrium equations. This method could assess the wall panel behavior with good accuracy and because of its simplicity it is proposed to be adoptable for non-linear dynamic studies.

The culmination of all these efforts was the development of CASHEW computer program [13, 14] by Folz and Filiatrault under the CUREe-Caltech Woodframe project. This program highlights the efficacy of cyclic analysis in light of full non-linear dynamic analysis and adopted a unified approach for both monotonic and cyclic analysis of WSW panels. This was done using the monotonic experimental data obtained from screw tests and modelling the screws as nonlinear orthogonal spring pair. Once the monotonic curve is obtained, the hysteresis behavior is obtained by using CUREE protocol and piece wise linear path defining rules. The results matched well with experiments. The backbone of all these modelling efforts was to characterize the screw behavior from screw component level tests and incorporate them in the numerical or analytical formulations. Now, in CFS research, the early efforts were by Fülöp and Dubina [15]. They performed screw connections tests in order to establish design criteria on seam and frame-sheathing connections in corrugated CFS wall panels. In an attempt to numerically model the wall panels using finite elements, they incorporated screw connection test data into the FE model. They found striking similarity between the experimental and simulated results. Xu and Martinez [16] have proposed a simple method to estimate the ultimate strength and associated lateral displacement of CFSSW panels. This method draws an analogy between eccentrically loaded bolt group and laterally loaded CFSSW panels and adopts Brandt's inelastic method for evaluating ultimate strength. The results were compared with contemporary experimental data and were found to be in good agreement. But this method doesn't predict the entire load-deformation history. Moreover, as reported by the authors the method predicts lateral strength more accurately than lateral displacement. The same authors developed a simplified numerical approach by modelling the CFSSW panel as equivalent sixteen noded orthotropic shell element [17] and incorporated it in SAP2000 software for assessing performance of a mid-rise CFS building under lateral loads. Although this method reduces the computational cost by modelling the whole panel as 16 node shell elements, the shear forces in the panels and internal forces in the studs are overestimated. Fiorino et.al[18] have performed several tests on screw connections with wood and gypsum sheathing for assessing the effect of different parameters like loading rate, sheathing orientation, sheathing edge distance and so on.

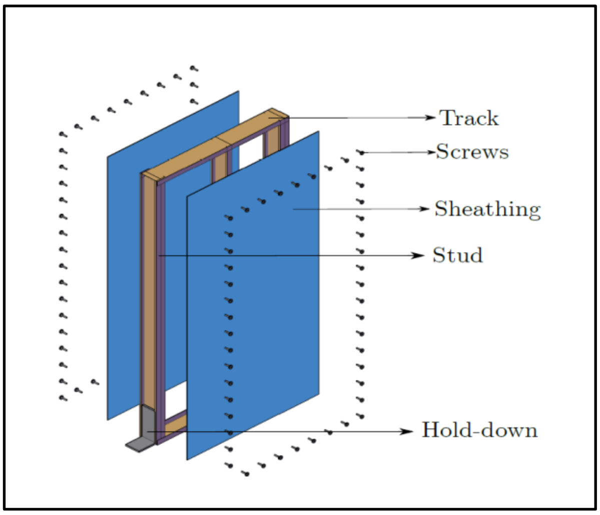


Fig. 1 Layout of CFS shear wall panel

With the data from screw tests schematized into relationship proposed by Richard and Abbott, they have also presented an analytical method for predicting the lateral load-displacement curve of CFSSW panel sheathed with oriented strand board on one side and gypsum board on other side [19]. The results were in good agreement with experiments. Buonopane et.al [20] have modelled the behavior of CFSSW panel in OpenSees [21] by a similar approach as in WSW panels. The studs and tracks are modelled as displacement based beam column elements, the sheathing as *RigidDiaphram*. The fasteners are modelled as *CoupledZeroLEngth* element with *pinching4* material with parameters

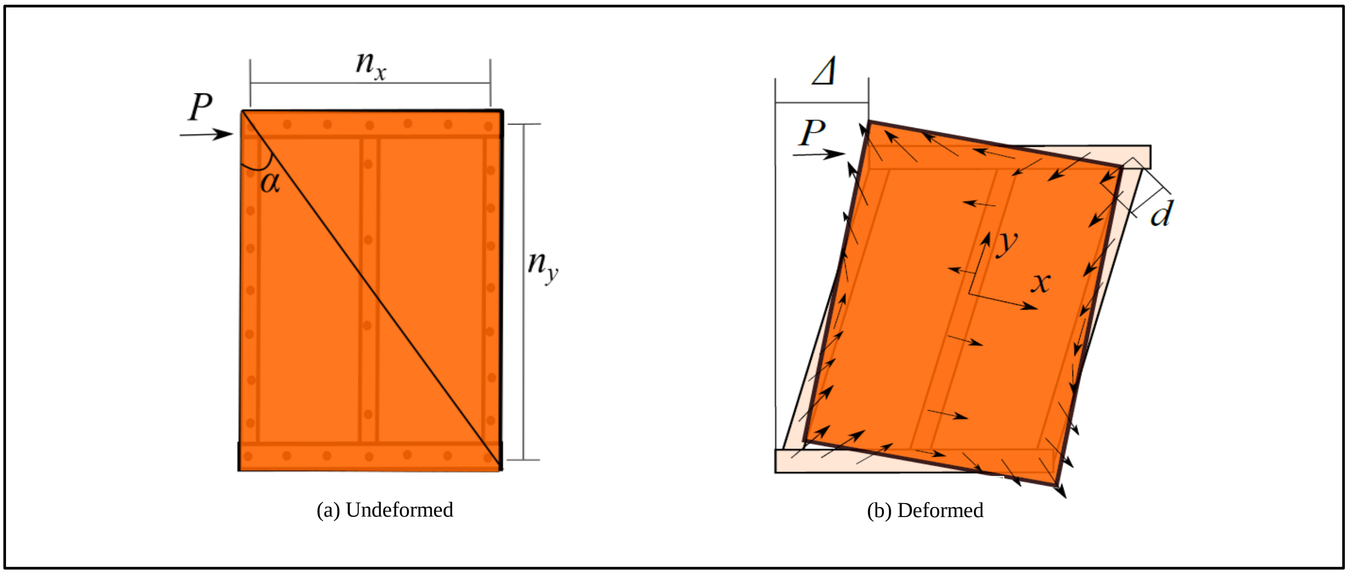


Fig. 2 Response of wall panel under lateral load action

calibrated to cyclic screw connection tests. The model reasonably captured the key characteristics of the cyclic load-displacement response. These models are efficient, in that, they fall between detailed finite element models and simple frame models and the only experimentally derived input is parameters of pinching4 material that are obtained from fastener cyclic tests. Also, these models give significant insight into fastener forces, stud forces and so on. However the model under predicts the cumulative energy dissipated with increase in number of cycles. Karabulut and Soyoz [22] modelled the CFSSW panel in SAP2000 by characterizing the screw test data to a revised Richard and Abbott model and modelling the fasteners as non-linear connectors.

The results matched well with experiments. However, revising the Richard and Abbott model for every type of sheathing to framing connections is cumbersome on trial and error basis.

The above study shows that procedures proposed in the literature involve either numerical simulation which is prone to convergence issues and cumbersome modelling in FEM software or simplified methods where there is a tradeoff between strength and deformation predictions. Therefore there is necessity for a simplified methodology to reliably analyze the lateral load displacement response of CFSSW panels, which can be easily adopted by researchers and practicing engineers for arriving at reliable prediction of strength and deformation capacities. This paper presents a purely mechanics based approach which is simple to implement, nevertheless reliable in it's prediction of the entire lateral load displacement response of the CFSSW panels with rigid sheathing materials such as OSB, calcium silicate board, gypsum wall board and so on. The methodology is implemented and verified against the experimental data for four different geometric and material configurations representing typical variations in CFSSW panels, adopted from the literature and the results are found to be in good agreement.

2. Methodology

The methodology adopted in the present approach is based on geometric relations between the CFS frame top displacement and the screw deformations and screw resistance developed thereof. The entire load-deformation history of CFSSW panel with rigid sheathing is discussed in two parts:

- Response till peak load
- Post-peak response

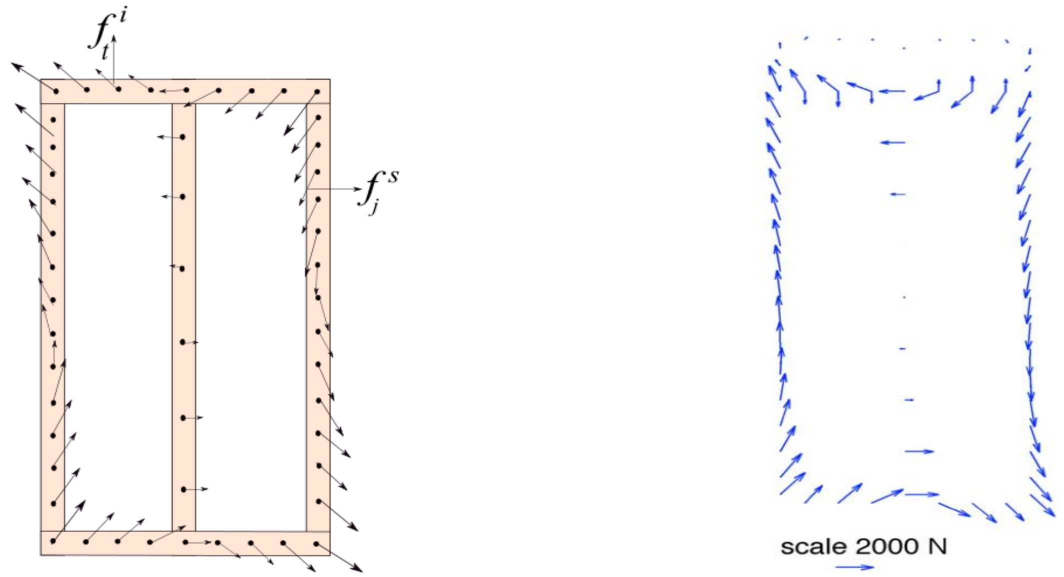
The idea behind such demarcation is primarily the change in deformation mode of the sheathing and subsequent change in geometric relations, as will be explained further.

2.1. Response till peal load

When a lateral load is applied to a wall panel, the steel stud frame distorts into parallelogram and the sheathing undergoes rigid body rotation. There is no significant deformation observed in the sheathing and is considered to remain rectangular. This relative displacement between the framing and sheathing causes shear force to be acting on the fasteners. The shear resistance provided the fasteners equilibrates the external load acting on the wall panel. The fundamental relationships in deriving the resistance provided by the fasteners had been adopted from Tuomi and McCutcheon [5] and McCutcheon [7]. However, the latter has adopted energy approach to solve for the resistance developed, but an equilibrium approach has been adopted in this study. The following are the assumptions made in this approach:

1. Frame distorts as a parallelogram and the sheathing retains its initial rectangular shape.
2. Sheathing undergoes rigid body rotation about its geometric center.
3. Sheathing is continuous from top of the frame to bottom.
4. The overall response of the wall panel is dictated by the response of the individual fasteners only and the shear deformation of the sheathing itself is negligible and hence the shear resistance developed thereof.
5. The screws are spaced evenly and symmetrically at the perimeter of the wall panel and in the field.

Assumption 4 is justified from the previous research[20][23] i.e. the stiffness at local fastener location is considerably lower compared to the overall diaphragm shear stiffness of the brittle sheathing panels(generally used for CFSSW such as wood based panels), for it leverage on the shear resistance of sheathing material. Therefore the global wall behavior is essentially dictated by local fastener behavior. Considering assumptions 1, 2 and 4, ductile sheathing materials such as steel sheets, are not in the scope of this paper. However simpler mechanics based methods that take tension field action into account are proposed earlier [24].



(a) Resultant force vector diagram of force exerted by Screws on CFS frame

(b) Resultant force vector diagram of force exerted by Screws on CFS frame

Fig. 3 Forces on wall panel at discrete fastener locations

Fig.2 shows the undeformed and deformed configurations of the wall panel. It is found from several experiments by past researchers that the corner screws deform the most. These deformations are approximately along the sheathing diagonals. Calculations by [7] show that, moderate change in this direction doesn't alter the performance of wall panels significantly. So for mathematical simplicity, it can be approximated that corner screws deform along sheathing diagonals as shown in fig.2. With this approximation, the deformation of all other screws can be expressed in terms of corner screw deformation (eq. 1 to 4). Let 'd' be the corner screw deformation along the sheathing diagonal, which makes an angle 'α' with the vertical side fig.2(a). The individual screw deformation is denoted by 'δ', with horizontal and vertical components as 'δ_X' and 'δ_Y' respectively. Now, from McCutcheon [7] the following relations can be established.

For the top edge:

$$\delta_x = d\ sin\ (\alpha) \tag{1a}$$

$$\delta_y = -\left(2\frac{i}{n_x} - 1\right) d \cos(\alpha) \tag{1b}$$

For the bottom edge:

$$\delta_x = -d\ sin\ (\alpha) \tag{2a}$$

$$\delta_y = -\left(2\frac{i}{n_x} - 1\right) d \cos(\alpha) \tag{2b}$$

For the left edge:

$$\delta_x = -\left(2\frac{j}{n_y} - 1\right) d \sin(\alpha) \tag{3a}$$

$$\delta_y = d\ cos\ (\alpha) \tag{3b}$$

For the right edge:

$$\delta_x = \left(2\frac{j}{n_y} - 1\right) d \sin(\alpha) \tag{4a}$$

$$\delta_y = -d\ cos\ (\alpha) \tag{4b}$$

Where,

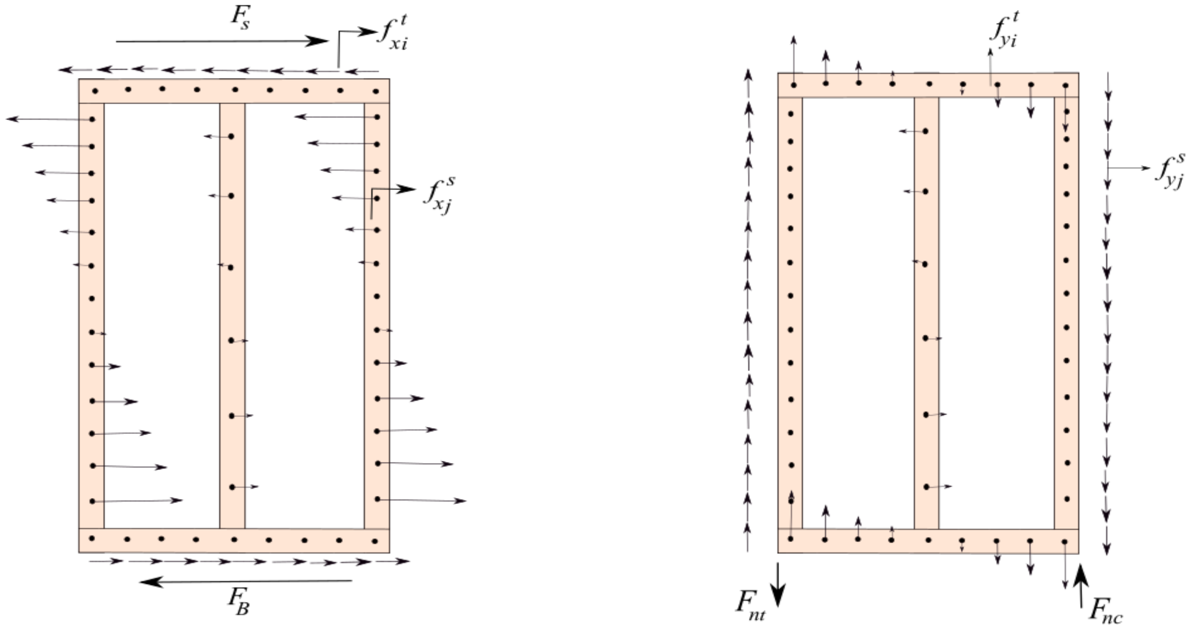
n_x is the number of screw spacings on tracks,

n_y is the number of screw spacings on studs,

i = 0,1,2,…. n_x

j = 0,1,2,…. n_y

Now, since the screws transfer the shear forces between the stud frame and the sheathing, the stud frame also experiences force in the same direction of the deformation of screws. These force vectors are shown in fig.3 (a). This fact is corroborated by observing the vector force diagram obtained using



(a) Horizontal components of fastener forces at each screw location

(b) Vertical components of fastener forces at Each screw location

Fig. 4 Force components on wall panel at discrete fastener locations

Computational analysis in OpenSees software by Buonopane et.al [20]. Therefore directions of the resultant vectors as per the proposed theory are similar to that of validated computational models. Now, resolving these vectors into horizontal and vertical components as shown in fig.4 (a) and (b). 'f_{xi}^t' and 'f_{yj}^s' are the horizontal components of screw force vector at ith location on the track and jth location on the stud respectively (fig.4 (a)). 'f_{yi}^t' and 'f_{yj}^s' are the vertical components of screw force vector at ith location on the track and jth location on the stud respectively (fig.4 (b)). F_s is the external shear force acting and F_B is the base shear developed. F_{nt} and F_{nc} are the reactions developed at the tension side and compression side of the wall panel respectively.

The proposed method adopts an algorithm, wherein the applied wall-panel top track lateral displacement (Δ) is given as the input. This lateral displacement (Δ) is related to the horizontal and vertical components of screw displacements using the simple geometric relationships as given in eq.1 through 4. From the resultant displacement, the resultant force developed at individual screw location is evaluated by characterizing the screw load displacement data to a 4th degree polynomial. Thereafter, by principles of equilibrium, the resistance developed in the wall panel is evaluated.

The global displacement of the top track 'Δ' is related to corner screw displacement 'd' as,

$$d = \frac{1}{2}\Delta\, sin\,(\alpha) \tag{5}$$

$$\delta_k = (\delta_{kx}^2 + \delta_{ky}^2)^{\frac{1}{2}} \tag{6}$$

Therefore, from eq.1, 2, 5 and 6, the resultant screw deformation at ith location on the top and bottom tracks is,

$$\delta_i^t = \frac{1}{2}\,\Delta\sin\alpha\,\left[\left(2\frac{j}{n_y}-1\right)^2\, sin^2\alpha + \,cos^2\alpha\right]^{\frac{1}{2}} \tag{7}$$

Similarly, from eq.3, 4, 5 and 6, the resultant screw deformation at jth location on the left and right tracks is,

$$\delta_i^t = \frac{1}{2}\,\Delta\sin\alpha\,\left[\left(2\frac{j}{n_y}-1\right)^2\, sin^2\alpha + \,cos^2\alpha\right]^{\frac{1}{2}} \tag{8}$$

Therefore, in eq.7 and 8, the individual screw displacement is expressed in terms of global wall lateral displacement. Now these screw displacements have to be related to screw force. Now we have to relate the screw deformation to screw force. This is established mainly by fitting a curve to the screw test data, so that screw force can be expressed as a function of screw displacement. Therefore at any instant the screw force is dependent only on the instantaneous screw displacement and is independent of the displacement history. As the general screw load displacement data follows a polynomial curve [25], a 4th degree polynomial is chosen to represent screw force as a function of screw displacement. Sometimes even 3rd degree polynomial also suffices, but in order to maintain consistency in implementation, a 4th degree polynomial is chosen. The assumed polynomial is written in the form,

$$f(x) = ax^4 + bx^3 + cx^2 + dx + e \tag{9}$$

The coefficients [a,b,c,d,e] vary for different screw and sheathing combinations. But once the coefficients are evaluated for a particular configuration, by appropriate curve fitting techniques, then force developed in the screws can be established in terms of screw displacement as a continuous function. Therefore, from eq.9 the force developed in each individual screw 'f_k' can be expressed in terms of individual screw displacement 'δ_k' as,

$$f_k(\delta_k) = a\delta_k^4 + b\delta_k^3 + c\delta_k^2 + d\delta_k + e \tag{10}$$

Therefore, in a displacement controlled loading, when the wall panel is laterally displaced by 'Δ', from eqs.7 and 8 we know the screw deformations (δ) in terms of 'Δ'. In eq.10, we have related screw deformations (δ) to force generated in the screws 'f_k'. Now the force developed at each discrete screw location has to be related to total shear resistance developed by the panel 'F_s'. This can be evaluated by considering the free body diagram of the wall panel with horizontal components of the screw forces as shown in fig.5. Considering the upper part of X - X, for the panel to be in equilibrium the lateral force should be equal to sum of the horizontal components of the screw forces located on upper part of the X - X. This can be written as,

$$F_s = \sum_{i=1}^{n_x} f_{xi}^{ut} + \sum_{j=1}^{\frac{n_y}{2}} f_{xj}^{urs} + \sum_{j=1}^{\frac{n_y}{2}} f_{xj}^{uls} \tag{11}$$

Similarly for the lower part of X - X, the base shear 'F_B' should be equal to sum

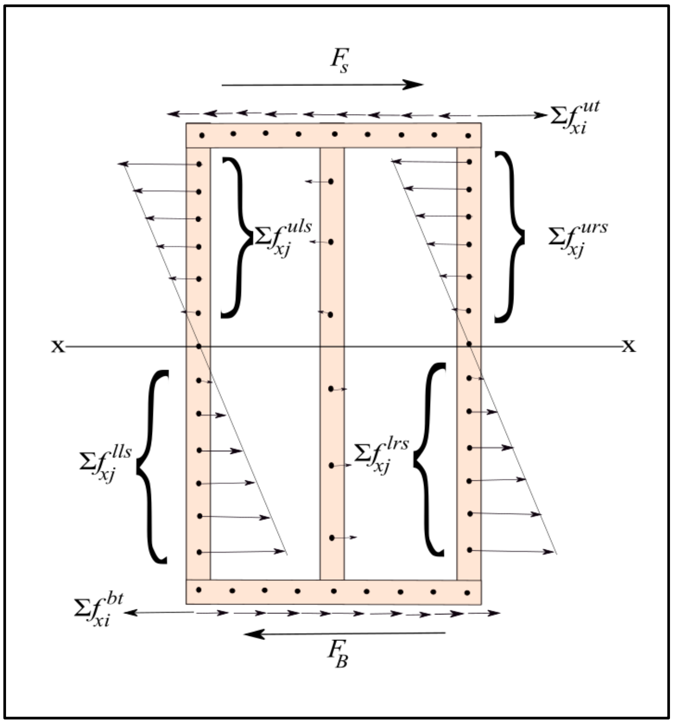


Fig. 5 Free body of frame showing horizontal components of the screw forces

of the horizontal components of the screw forces located on lower part of the X - X. This can be written as,

$$F_B = \sum_{i=1}^{n_x} f_{xi}^{bt} + \sum_{j=\frac{n_y}{2}}^{n_y} f_{xj}^{lrs} + \sum_{j=\frac{n_y}{2}}^{n_y} f_{xj}^{lls} \tag{12}$$

Where in eq.11 and eq.12,

f_{xi}^{ut} – Horizontal component of screw force located at the i[th] position on the upper track,

f_{xj}^{urs}- Horizontal component of screw force located at j[th] position on the upper right stud.

f_{xj}^{urs}- Horizontal component of screw force located at j[th] position on the upper left stud

f_{xj}^{bt}- Horizontal component of screw force located at j[th] position on the bottom track

f_{xj}^{lrs}- Horizontal component of screw force located at j[th] position on the lower right stud

f_{xj}^{lls}- Horizontal component of screw force located at j[th] position on the lower left stud

Therefore, evaluating F_B for every displacement step gives the resistance developed in the panel. The eq.1a to 8 are implemented and were found to reliably capture the behavior of the CFSSW panel till peak load. After the peak load, the evaluated resistance would be over-estimated. This is because of the inherent assumptions like small displacements of screws that are made in deriving the above relationships. But in reality the panel undergoes large deflection beyond peak load. This is explained in the following section.

2.2. Post peak response

It has been reported by many researchers [26, 27, 28, 29] that after failure of wall-panel that sheathing was observed to have underwent overturning movement. That means the center of rotation has shifted from geometric center of the sheathing towards the corner. This is also corroborated by the evaluation of instantaneous center of rotation for calculating the ultimate load in the analytical approach proposed by Xu and Martinez [16]. The instantaneous center of rotation was offset from the center of screw group at the ultimate strength level. Therefore, for evaluating the response after the peak load, the center of rotation is assumed to have shifted from center of panel to corner of the panel as shown in fig.6. 'O' is the center of rotation till peak load and 'O' is the center of rotation beyond peak load. Now the geometric relationship between post peak global lateral displacement 'Δ_{pp}' and the individual screw displacement 'δ' can be expressed directly as,
For track screws,

$$\delta_i = \delta_{peak} + \frac{\Delta_{pp}}{p_i} \tag{13}$$

Where,

δ_i is the displacement of i[th] screw on the track,

p_i is the position of i[th] screw from the corner O',

δ_i is the displacement of i[th] screw on the track,

Δ_{pp} is the post peak displacement of the wall panel,

s is the screw spacing on the track.

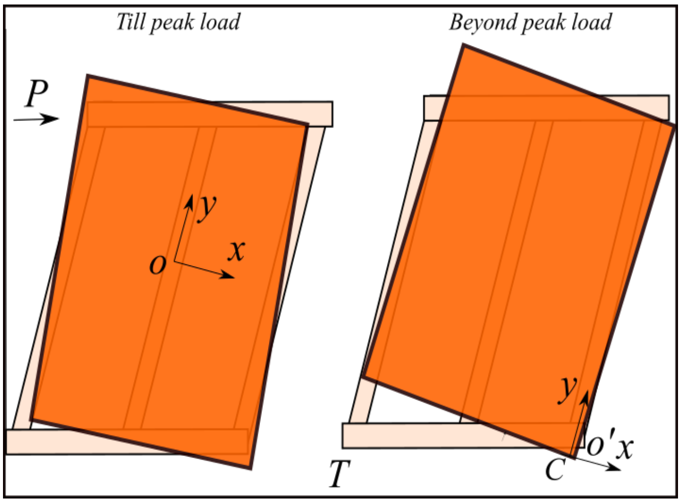


Fig. 6 Peak and post peak deformation patterns (exaggerated)

This process is repeated for every global lateral displacement step and force developed is evaluated in the same way as described in the previous section, eq.10 to 12. From eq.13, it can be seen that the corner screw displacements keep on monotonically increasing leading to either edge tearing of the panel at the tension side (T) or the board crushing on the compression side (C) of the board, incapacitating the screws at these locations to offer shear resistance. So, in the algorithm, the screws that cross their ultimate displacement limit are traced and their resistance is made zero. Similar criteria applies to the screws on studs also. Finally the program stops when more than 40% screws are failed on the tracks indicating the onset of instability and attainment of ultimate resistance. The whole approach can be summarized as shown in fig.7.

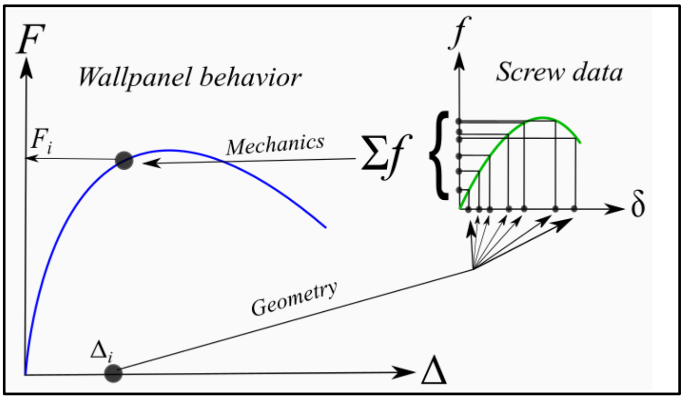


Fig. 7 Summary of the approach

It is to be noted that the eq.11 is presented for one sheathing panel. But if multiple panels are used, this approach has to be applied on each individual sheathing panel and the resultant lateral resistance is simply a sum of lateral resistance of each individual panels. The multiple panels may be placed either

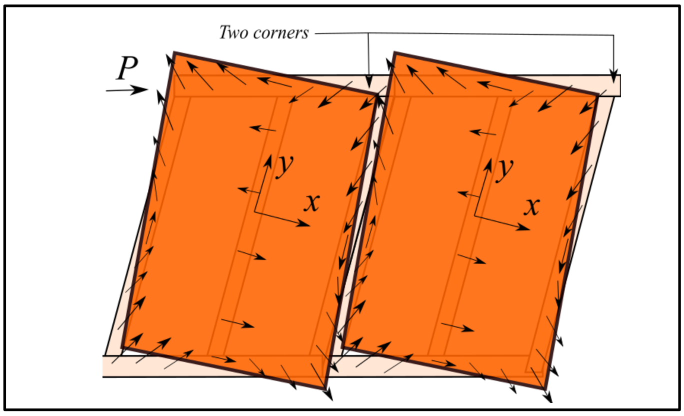


Fig. 8 Wall panel with two sheathing boards

on one side or on both sides, still the approach is valid. The basic geometric relations (eq.1 to eq.8) remain valid irrespective of number and size of the wall panel. However, while applying eq.5, to multiple wall panels, as many corner screws as that of sheathing boards have to be considered. For example, as shown in fig.8, there are two sheathing boards and hence two corner screws have to be considered. The displacement of other screws in a particular sheathing board have to be expressed in terms of corresponding corner screws and finally the resultant force can be obtained by simple linear addition of 'F_s' (eq.11) of each individual panel.

Thus, the proposed algorithm for assessing the response of CFSSW panels has been described. The approach is purely mechanistic, involving only principles of mechanics and geometric relations. There are no numerical methods involved, thereby alleviating the complex modelling and convergence issues confronted thereof. The proposed method is incorporated into a computational framework CFS-RAMA (Racking Analysis through Mechanistic Approach), which can be used for any configuration of wall panels with rigid sheathing. The following section describes the results obtained from this computer program.

3. Implementation and results

In the past two decades there has been a significant increase in the experimental research on lateral behavior of CFSSW panels. Many researchers have done experimental studies on different sheathing materials and different configurations of CFSSW panels. There are also experimental studies on component level screw tests in order to investigate the sheathing to screw interaction behavior for different sheathing materials, thickness, screw sizes, sheathing orientation, edge distance, loading protocols and so on. The present study extracts the data from four different configurations of CFSSW panels across the literature, whose screw tests and full scale wall panel tests are performed and reported [30]. The four configurations are chosen such that, they account the wide variety of configurations used in the industry. The implementation details and the results are presented in this section. The screw and full scale wall panel test data has been collected from the experiments of the following researchers in the literature,

1. Padilla-Llano et.al. [31, 32]
2. Nithyadharan and Kalyanaraman [33, 26]
3. L.Fiorino et.al. [18]

Table 1
Configurational details

Config. No.	Name	Sheathing type	Overall dimensions (mm)	Description
C-1	Padilla-Llano et.al [31, 32]	1067x2413mm, 11mm thick Oriented strand board one on one side	1067 x 2413	The sheathing is attached to frame (1.37 mm thick) using #8 flat head fasteners
C-2	Nithyadharan and Kalyanaraman [33, 26]	1200 x 2400, 10mm thick Calcium silicate boards, one each on both sides	1200 x 2400	The calcium-silicate boards have been attached to the frame using 4mm screws, paced at 150mm c/c on the perimeter and 300mm c/c in the field studs.
C-3	Nithyadharan and Kalyanaraman [33, 26]	600 x 2400, 10mm thick Calcium silicate boards, 2 no's placed adjacent to each other on both sides	1200 x 2400	The boards were placed side by side on both sides and are attached to framing with screw spacing of 150 mm.
C-4	L.Fiorino et.al [18]	Exterior panel: 1200 x 2500 mm, 9mm thick OSB, 2 no's placed side by side. Interior panel: 1200 x 2500, 12.5mm thick GWB, 2 no's placed side by side	2400 x 2500	The sheathing panels are placed side by side and are screwed to the frame at 150mm spacing on the perimeter and 300mm spacing on the field.

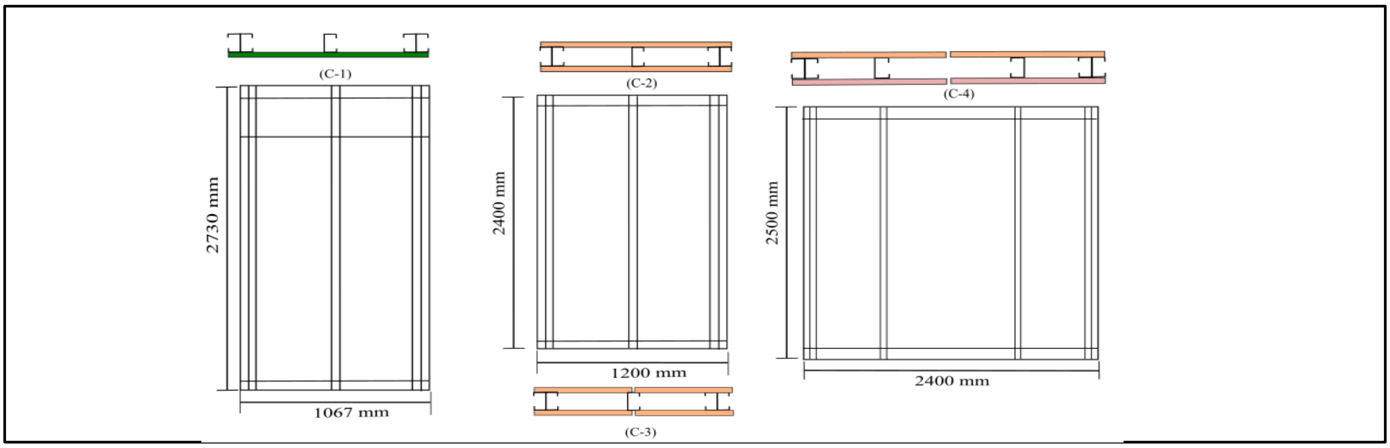
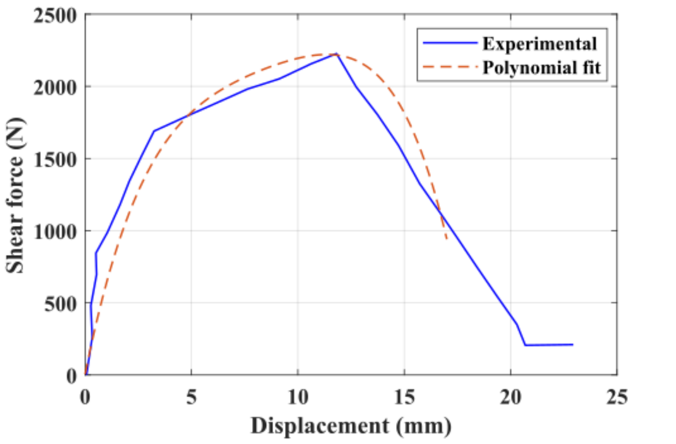
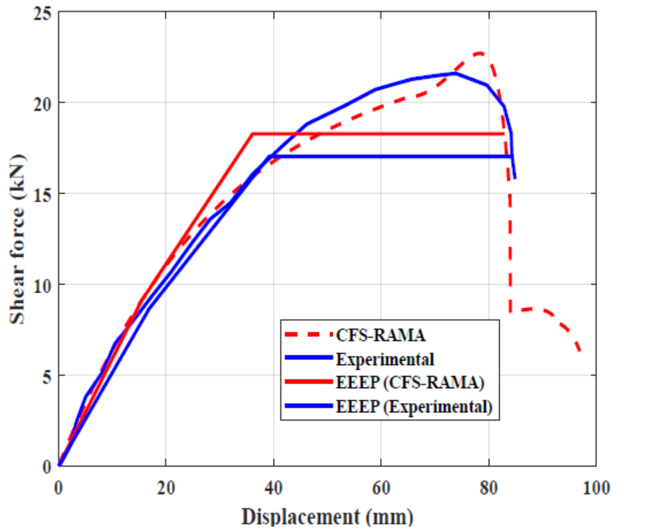


Fig. 9 Configurations considered for validation

The configurational details of these panels are outlined in the table.1. It can be observed that the materials used for sheathing and dimensions of the wall panel vary significantly from experiment to experiment. The dimensions and layout of these panels can be seen in fig.9.



(a) Polynomial fitted to screw test data from Padilla-Liano et.al [31,32]



(b) Comparison of experimental data vs obtained response for C-1

Fig. 10 Configuration-1, polynomial fitting and wall panel response

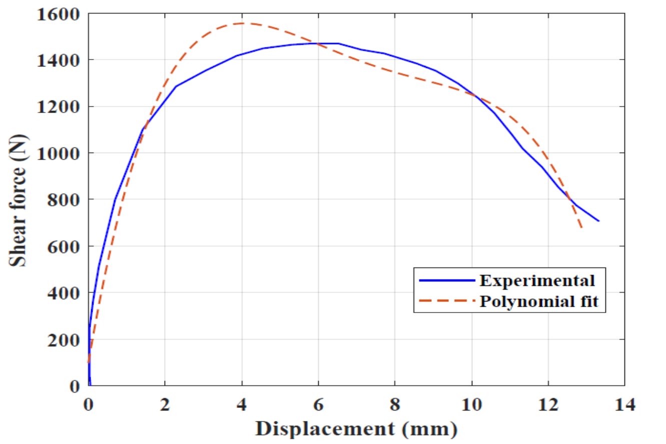
3.1. Configuration-1

The screw test data and the corresponding wall panel experimental data under monotonic loading has been extracted from Peterman et al [32] and Padilla-Llano et al. [30]. The configuration of the panel is as shown in fig.9(C-1) and table. 1. The wall panel is 2740mm high and 1220mm wide framed using back-to-back 600S162-54 CFS members. The framing is interconnected through #10 screws. This is a ledger frame with 1200T200-97 track that connects the wall to the floor diaphragm and is fastened to the vertical members at the top of the wall. On one side of the frame, OSB panel is attached and the ledger track on the opposite side. Two Simpson Strong-Tie S/HDU6 hold downs are connected to the bottom of the chord studs using #14 hex-head fasteners. Two 15.875mm (5/8in.) bolts serve as shear anchors. The single screw test data has been adopted from Peterman et al [32]. They have performed a series of screw tests in order to characterize the hysteretic behavior of stud to sheathing connection subjected to in-plane shear.

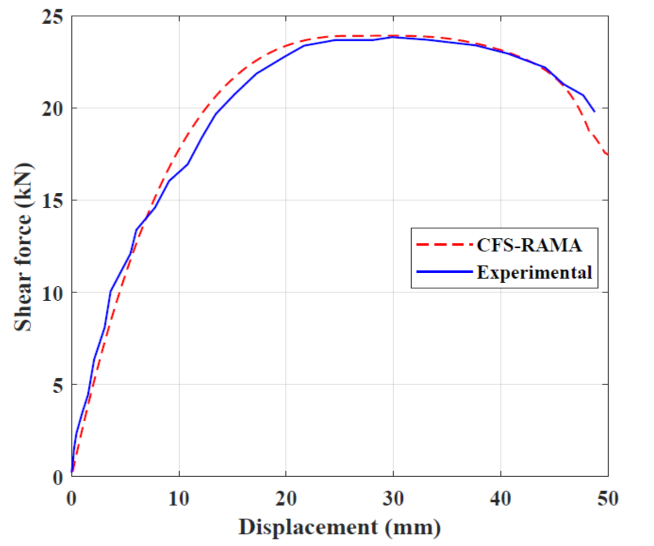
In order to implement the proposed method, the screw test data for 11mm OSB and 1.37mm studs has been extracted. A 4th degree polynomial is fit for the screw data using least squares and also equating the area under the curve. The fitted polynomial is shown in fig.10 (a). The proposed method is applied and the results obtained are shown in fig.10 (b). The Equivalent Energy Elastic Plastic (EEEP) plots are also shown. It can be seen that the proposed method captured the response of this configuration with fair accuracy. Moreover, the post peak response also matches well with the experiments. This validates the algorithm adopted for post peak load response. The peak load is overestimated but the ultimate loads and displacements are well captured. Therefore, this method can be used to predict the response of wall panels with similar configurations.

3.2. Configuration-2

The screw test data and corresponding wall panel test data for this configuration has been adopted from Nithyadharan and Kalyanaraman, see C-2 in table.1 and fig.9. Nithyadharan and Kalyanaraman have performed screw tests [33] and corresponding wall panels tests [26] on calcium silicate boards of different thickness. The screw test data corresponds to 3.9mm diameter screw drilled to 10mm thick calcium silicate board with edge distance of 25mm. The dimensions of the wall panel are 1200mm x 2400mm. In this configuration, 10mm thick calcium silicate boards of dimensions 1200mm x 2400mm are attached, one on each side of the steel framing. The screws are equally spaced at 150mm on the perimeter and 300mm on the interior studs. The proposed method is applied on this configuration and the results obtained are found to be in close comparison with experimental data, as shown in fig.11. Fig.11 (a) shows the 4th degree polynomial fitting and Fig.11 (b) compares the full scale wall panel test data with the obtained response. It can be seen that peak and post peak responses are also captured well. The initial stiffness and the peak load are also captured accurately. Therefore it can be concluded that the proposed method is well suited for similar configurations of wall panels.



(a) Polynomial fitted to screw test data from Nithyadharan et.al [26]



(b) Comparison of experimental data vs obtained response for C-2

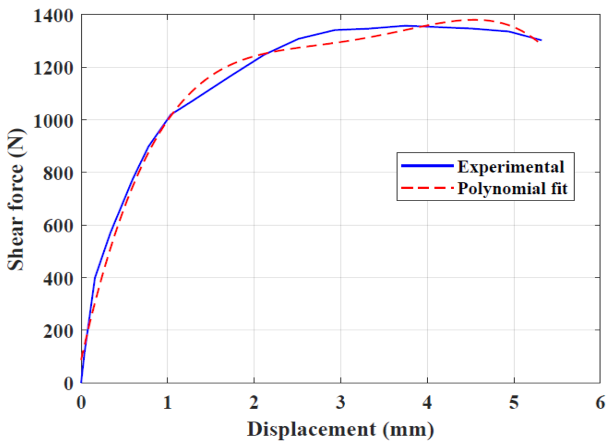
Fig. 11 Configuration-2, polynomial fitting and wall panel response

3.3. Configuration-3

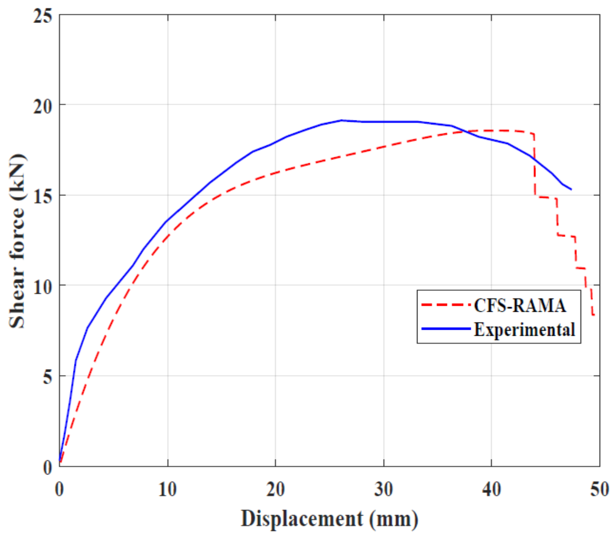
The screw test data and corresponding wall panel test data for this configuration has also been adopted from Nithyadharan and Kalyanaraman, see C-3 in table.1 and fig.8. However, the screw test data in this case corresponds to 3.9mm diameter screw drilled to 10mm thick calcium silicate board with edge distance of 10mm [33]. This configuration consists of 2 no's of 600mm x 2400mm calcium boards placed side by side on both sides of the wall framing. The overall dimension is 1200mm x 2400mm. The sheathing panels are attached to the frame with screws with spacing 150mm all around. The proposed method is applied to this configuration and the results are shown in fig.12. It can see that the results are in good agreement with the experimental values. But the initial stiffness is a little overestimated and also the peak load is little underestimated. This may be due to the contact between the adjacent sheathing boards and crushing against each other. Also the screw test data may not represent the average experimental values. However, this method reasonably assess the behavior of such configurations.

3.4. Configurartion-4

The screw test data and corresponding wall panel test data for this configuration has also been adopted from L.Fiorino. [18], see C-4 in table.1 and fig.9. This configuration consists of 2 no's of 1200mm x 2500mm, 9 mm thick OSB sheathing on the exterior side, placed adjacent to each other and 2 no's of 1200mm x 2500mm, 12.5 mm thick GWB sheathing on the interior side, placed adjacent to each other. The overall wall panel dimensions are 2400mm x 2500mm. The proposed method is applied to this configuration and the results are shown in fig.13. It can be seen that the results match well with the experimental data. The peak and post peak responses are captured well. This implies that this method can be used for estimating the response of wall panels of this configuration.



(a) Polynomial fitted to screw test data from Nithyadharan et.al [26]



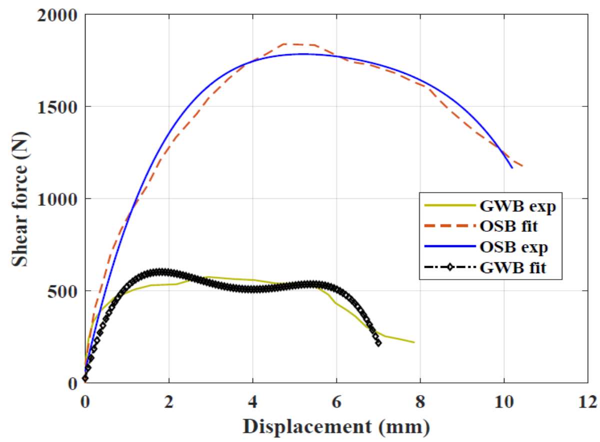
(b) Comparison of experimental data vs obtained response for C-3

Fig. 12 Configuration -3, polynomial fitting and wall panel response

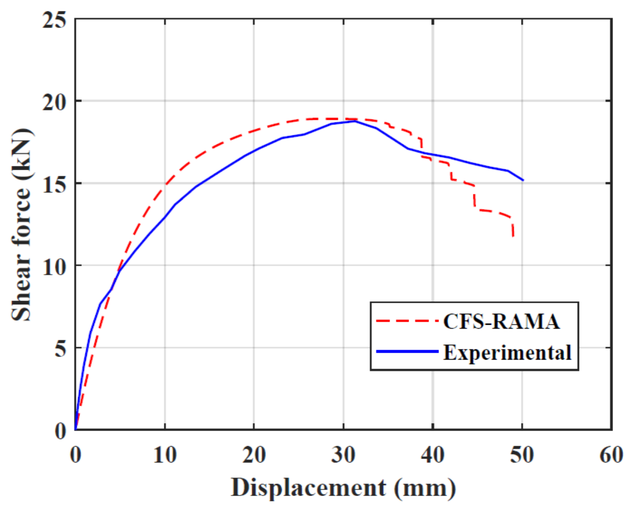
The design parameters of the four test configurations are estimated using EEEP method [34] and compared against their experimental counterparts. The results are summarized in Table.2. Therefore it can be seen that the proposed method reliably captures the behavior of the CFSSW panels under monotonic lateral load. The method is tested against various configurations of sheathing boards and wall panel dimensions and the results were satisfactory. The proposed method is simple and can be easily adopted in engineering practice. It can be conveniently programmed in spreadsheets also. The only input is the experimental data from component level screw tests.

3.5. Ductility

The ductility of shear wall panels comes from the sheathing – frame connections. The response reduction factor (R) given in ASCE 7-16 is 6 for CFS shear walls with steel sheet and wood based sheathing, and 2.5 for shear walls with other brittle sheathing materials. This shows the energy dissiparion capability of these systems based on their ductility is reliable for earthquake resistant design. However, it is to be noted that the ductility of CFSSW panels is completely dependent of the sheathing and not on the frame. The frame can possibly trigger a brittle mode of failure by chord studs buckling under compression. But this can be avoided by properly designing the chord studs using capacity design principles. Another key component that enbles the development of full shear strength is the hold downs. The hold downs are to be designed to resist the tensile force developed in the chord studs at the point of peak shear resistance.



(a) Polynomial fitted to screw test data from Della Corte et.al [18]



(b) Comparison of experimental data vs obtained response for C-4

Fig. 13 Configuration-4, polynomial fitting and wall panel response

3.6. Industrial application

The proposed method can be standardized if the sheathing manufacturers can perform in-house screw tests. The whole screw test data can be transferred to the customers just in the form of the 4th degree polynomial coefficients [a b c d e] (Refer eq.10) for a given sheathing material thickness and screw dia. Therefore the sheathing manufacturers can standardize the polynomial coefficients and just specify them for each sheathing thickness and corresponding screw diameter, then the monotonic pushover curve for any configuration of CFSSW panel can be assessed by the designers with the proposed method. Therefore the method can be readily used in the analysis of CFSSW panels.

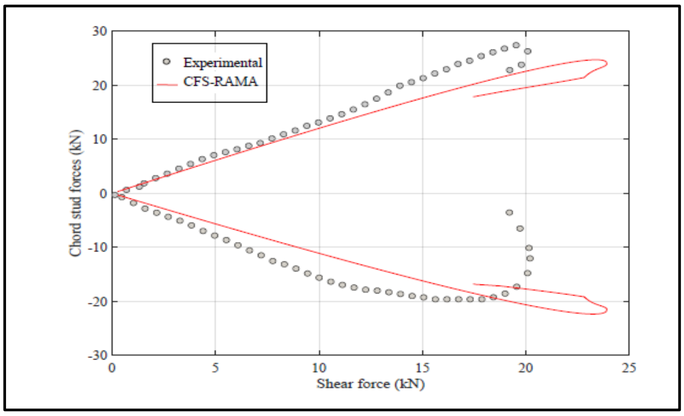
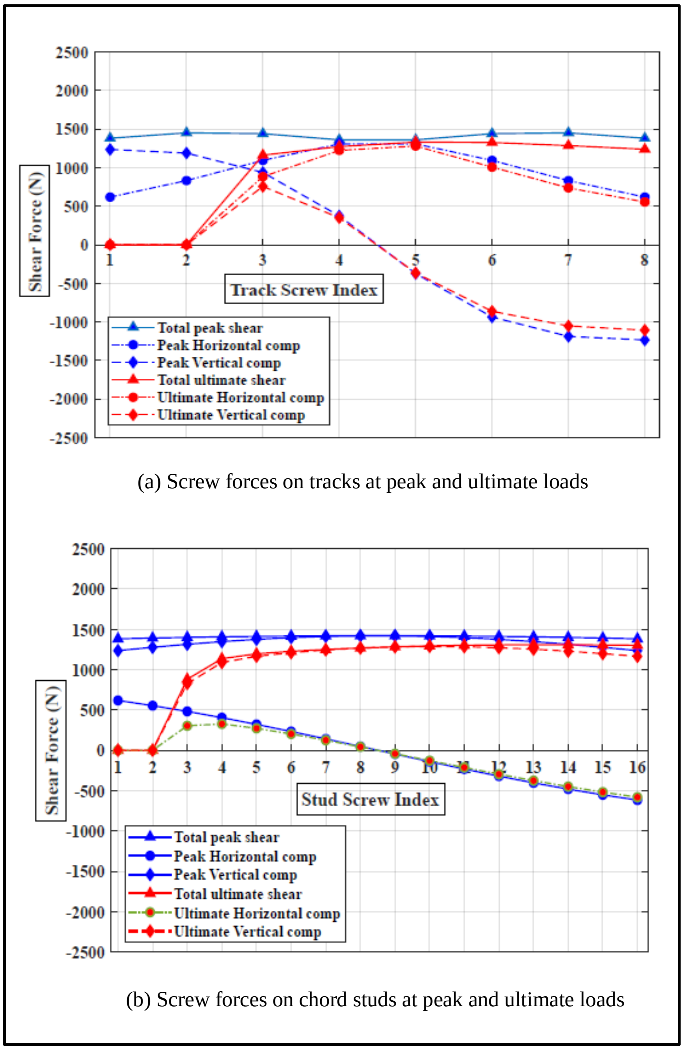


Fig. 14 Chord stud forces on the tension and compression side of C-2 wall panel



(a) Screw forces on tracks at peak and ultimate loads

(b) Screw forces on chord studs at peak and ultimate loads

Fig. 15 State of fasteners at peak and ultimate loads of C-2 wall panel

The proposed approach also gives a significant insight to the forces developed in the chord studs and also state of the fasteners at any given point on the load-displacement curve of the wall panel. Fig.14 (a), (b) shows the state of fasteners at the peak and ultimate load points on tracks and studs of the C-2 wall panel. Also from fig.4 (b), the force developed in the chord studs and hold downs can be easily established. Fig.15 shows the axial load developed in the tension and compression side chord studs and compares with experimental values proposed by Nithyadharan and Kalyanaraman [27]. The predicted peak axial load value is, $\pm$23kN which is close to experimental value of, $\pm$20kN. This difference may be attributed to highly non-linear load sharing that happens between chord studs and sheathing, through screws connecting them near the region of peak load. So, the proposed method fits in the Performance Based Seismic Design (PBSD) paradigm by enabling the designer to assess the fastener states at any point on the load-displacement curve of the wall-panel, thereby incorporating performance parameters.

4. Limitations

The predictive capabilities of the proposed method for different sheathing materials and different wall configurations are demonstrated in the preceding section. However the method has few limitations which can be addressed by further research to improve its accuracy and generality of application. They are:

1. The proposed method is valid only under the rigid sheathing board assumption and does not include the shear deformation of the sheathing material. However for most of the rigid sheathing materials currently being used, this assumption does not affect the accuracy of prediction. This has been demonstrated for wood based sheathing materials such as OSB and also other sheathing materials like calcium silicate board and gypsum wall board. However this method cannot be applied for ductile sheathing materials like steel sheet and so on. However similar mechanic based methods have been proposed for steel sheet sheathed wall panels.
2. The proposed method does not include the slippage of bottom tracks and uplift of hold downs. The underestimation of ultimate displacements may be attributed to this exclusion. However this can be included using some empirical relationships standardized from test data.
3. When two sheathing boards are used on the same side of the wall panel, adjacent to each other some interaction takes place between them, this method does not account for that interaction. However, in spite of that, the error in prediction is within tolerance

5. Conclusion

The present paper presents a simplified mechanistic displacement based approach which can reliably capture the lateral behavior of CFSSW panels with rigid sheathing materials, subjected to monotonic lateral load. The approach involves relating the global wall panel lateral displacement to individual screw displacements using geometric relations. The individual screw displacements are related to screw forces by fitting a 4th degree polynomial to the screw test data. The individual fastener forces are related to wall panel resistance using simple mechanics. This alleviates the detailed finite element modelling and convergence issues confronted thereof. There is no numerical solution technique as such used for solving the equilibrium equations. The approach is rather direct. The formulation has been explained in detail and also validated against four different wall configurations from the literature. The details of the wall panel configurations has been outlined and the comparison between the experimental and calculated response has been presented. The configurations vary significantly viz. the sheathing materials, the dimensions of the wall panel and the thickness of the sheathing materials. The design parameters estimated using EEEP method are also compared against corresponding experimental values and found in good agreement. The method seems to give a reliable estimate of the wall panel behavior for a variety of configurations, dimensions and sheathing materials used. The error is within tolerance and conservative. The predicted values of yield and peak displacement of configuration-3 are significantly higher compared to their experimental counterparts possibly due to sheathing-sheathing interaction on the same side. The proposed method has been demonstrated to be robust and reliable for assessing the lateral load behavior of CFSSW panels with rigid sheathing under monotonic loading. The use of the proposed method in practical shear panel design is highlighted.

Table 2
Comparing the EEEP design parameters for all the configurations

	Yield Strength (f_y) (kN)	Yield Displacement (Δ_y) (mm)	Stiffness (K_e) (kN/mm)	Peak Strength (f_p) (kN)	Peak Displacement (Δ_p) (mm)	Ultimate Load (f_u) (kN)	Ultimate Displacement (Δ_u) (mm)
Config.1 Experimental	16.1	6	2.7	18.8	31.3	15.1	50.2
Config.1 Proposed method	19.7	8.8	2.2	18.9	29.3	15.1	43.2
Error Proposed – experimental	3.5	2.8	-0.4	0.1	2	0	-7
Config.2 Experimental	21.8	8.3	2.6	23.8	29.9	19.8	48.7
Config.2 Proposed method	23.3	10.3	2.3	23.9	29.7	19.1	48
Error Proposed – experimental	1.5	2	-0.3	0.1	0.2	-0.7	-0.7
Config.3 Experimental	17	5.7	2.9	19.1	26.1	15.3	47.4
Config.3 Proposed method	19.1	11.3	1.7	18.6	40.7	14.9	45.3
Error Proposed – experimental	2.1	5.6	-1.2	-0.5	14.6	-0.4	-2.1
Config.4 Experimental	20.1	39.1	0.5	21.6	73.8	17	84.3
Config.4 Proposed method	21.3	36.1	0.6	22.7	78.3	18.3	82.9
Error Proposed - experimental	1.2	-3	0.1	1.1	4.5	1.3	-1.4

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