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Recommendations for design of sheathing bracing systems for slender cold-formed steel structural members

Sivaganesh Selvaraj, Mahendrakumar Madhavan *

Department of Civil Engineering, Indian Institute of Technology Hyderabad, Kandi, Sangareddy 502 285, Telangana, India

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ABSTRACT

The use of sheathing material as a structural component in the Cold-formed steel (CFS) construction holds great potential for stability of the structure and savings in the construction cost. However, there is no robust design method to account the structural contribution of sheathing boards. This paper presents the experimental results of 107 (including unsheathed and sheathed) CFS wall frame studs subjected to out-of-plane loading to study the feasibility of using gypsum board as a structural bracing sheathing material. The test parameters include various shapes and slendernesses of the CFS frame stud, thickness of the sheathing board and spacing between the sheathing bracing connections. The out-of-plane loading is applied as it causes lateral torsional buckling in the CFS structural members thereby creates pull-through failure at the sheathing bracing connections. Moreover, the suitability of the current AISI and Eurocode specification for the design of sheathing braced CFS structural member is studied. The experimental results indicate that the lateral buckling of the symmetric shaped (against the loading axis) CFS wall frame studs can be inhibited by gypsum sheathing. Whereas most of the singly symmetric and point symmetric CFS studs exhibit lateral torsional buckling and biaxial bending, respectively, due to the inadequate bracing effect of gypsum sheathing resulting in pull-through failure at the sheathing bracing connections. Therefore, a set of limitations for the use of gypsum sheathing as a structural bracing is suggested in the form of a generalized design parameter. Finally, the experimental results indicates that the design strength of the CFS wall frame stud can be increased in the range of 39% to 595% based on the shape and slenderness.

1. Introduction

Cold-formed steel structures are fast becoming a common material of choice for construction either as a complete structure or an internal partition due to its advantages such as high strength-to-weight ratio, resistance to corrosion, simple erection and demolition procedures and reusability. Despite the increase in the use of CFS structural member in construction activities, there is a dearth of erection and design guidelines that considers the failure modes of the various shapes of the CFS structural members and associated connections. Particularly, there are no design guidelines to optimize the structural member (built-up cross sections) or one that considers the structural effect of inherent constructional elements such as sheathing board (external cover) leading to cost effective construction. One such problem recognized by the industry is the design of bracing systems (steel or sheathing board) for CFS wall frames (Fig. 1a). The CFS wall frame studs have various shapes [C shaped channel, Z shaped and hat-shaped cross-sections – Fig. 2 (a-c)] and are typically slender due to its dimensional requirements (height and thickness of wall) and therefore requires additional bracing for stability. However, there are no robust design guidelines for the design of bracing systems for CFS wall frames. Hence, this investigation endeavors to provide preliminary suggestions for the design of sheathing braced slender CFS structural members.

2. Background on bracing design for CFS structural members

In general, the bracing systems for the structural members can be classified into two categories; torsional and lateral bracing, each of which restrain the lateral movement and twist of the structural member due to buckling. An effective bracing system restrains both the lateral movement and the twist of the cross-section [40]. In the case of slender CFS structural members subjected to axial compression or out-of-plane loading, the vulnerability of occurrence of torsional buckling (lateral-torsional and flexural torsional buckling - both of which lead to lateral displacement and twist) is high. Hence, the bracing system for CFS wall frame stud should be such that it restraints both twist and lateral displacement.

The bracing systems for CFS wall frame studs can be of two types: (i) conventional steel bracing (Fig. 1a), which will significantly increase the steel consumption thereby the cost of construction; (ii) sheathing

* Corresponding author at: Department of Civil Engineering, Indian Institute of Technology (IIT) Hyderabad, Kandi, Sangareddy 502 285, Telangana, India.
E-mail addresses: ce13p1009@iith.ac.in (S. Selvaraj), mkm@iith.ac.in (M. Madhavan).

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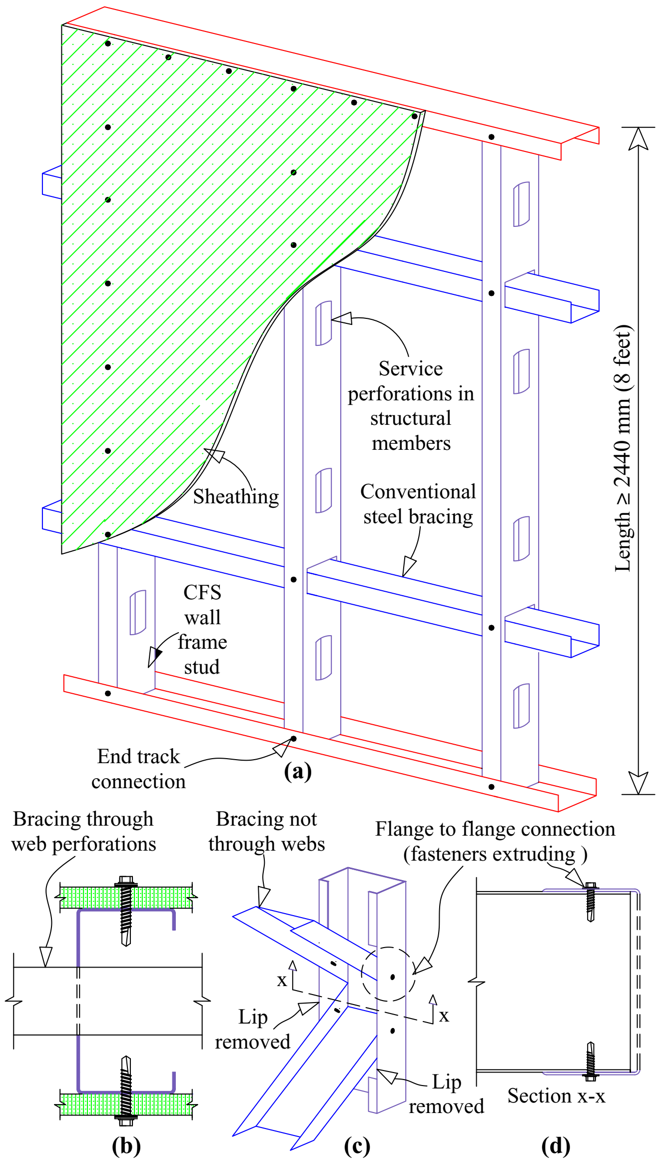

Fig. 1. (a) Cold-formed Steel wall frame stud with sheathing; (b) Provision of steel bracing through web of the CFS wall frame stud; (c) Provision of steel bracing attached at the flanges of the CFS wall frame stud; (d) Protruding screw connection.

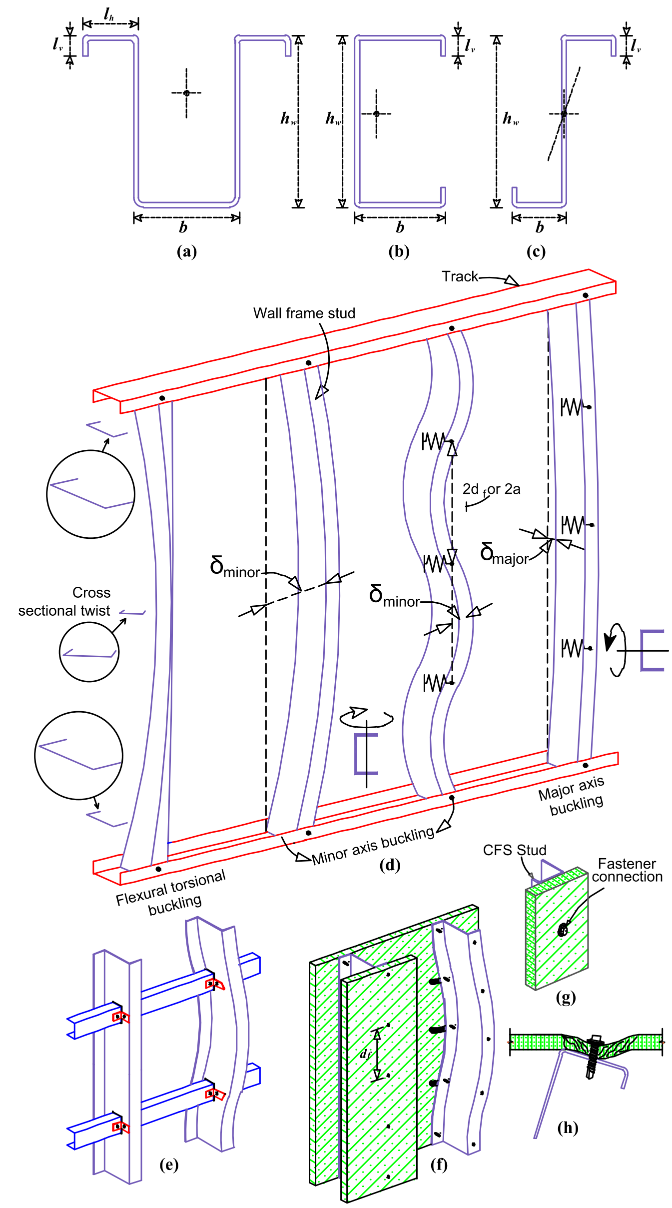

Fig. 2. (a) Inverted or reversed CFS hat section; (b) CFS Channel section; (c) Point-symmetric (Z) shaped CFS section.

board bracing (Fig. 2f) [currently being investigated by various researchers [5,9,14–16,30–36,39,41, 12] - bracing design by considering the inherent structural contribution by sheathing boards. In addition to increasing the cost of construction, the conventional steel bracing system has several disadvantages. The provision of additional steel bracing requires perforations in the webs of the CFS structural members for continuity (Fig. 1a and b), which will further increase the vulnerability due to instability and reduce the design strength significantly. This apprehension has led to the provision of excessive amount of bracings (over conservative design) in the CFS wall frames as a preventive measure to avoid failure due to instability. Fig. 3 shows the construction of residential building using cold-formed steel structures with excessive amount of steel bracings due to the unavailability of reliable design guidelines based on experimental test results. The reason for installing the conventional steel bracings through the webs (perforations - Fig. 1b) of the CFS wall frame studs rather than having connected at the flanges (without perforations – Fig. 1c and d) is due to the fact that the protruded screw connection for bracings at the flange obstruct the attachment of sheathing boards at the surface at the wall panels. It is also necessary that the lip of the CFS wall frame stud has to be removed when the steel bracing is connected to the flanges, which will increase the vulnerability of local buckling as shown in Fig. 1c. The above blemishes of conventional bracing system indicates an imperative need for a new bracing system.

Although, the sheathing board bracing system which will eliminate the need for additional steel bracings, the design of sheathing based bracing system is undoable since the development of design guidelines is at a nascent stage (AISI [5,19–29]. To be specific, the strength and stiffness of the sheathing bracing connection against the various failure modes of the CFS wall frame stud is unknown and hence, it is impossible to adopt the sheathing braced design procedure suggested by AISI [2]. The term sheathing bracing connection means that the CFS wall frame stud and sheathing boards are connected by self-drilling screws (Fig. 2g). The available literature indicates that some of the conventional sheathing boards made of powdery materials with less fiber content

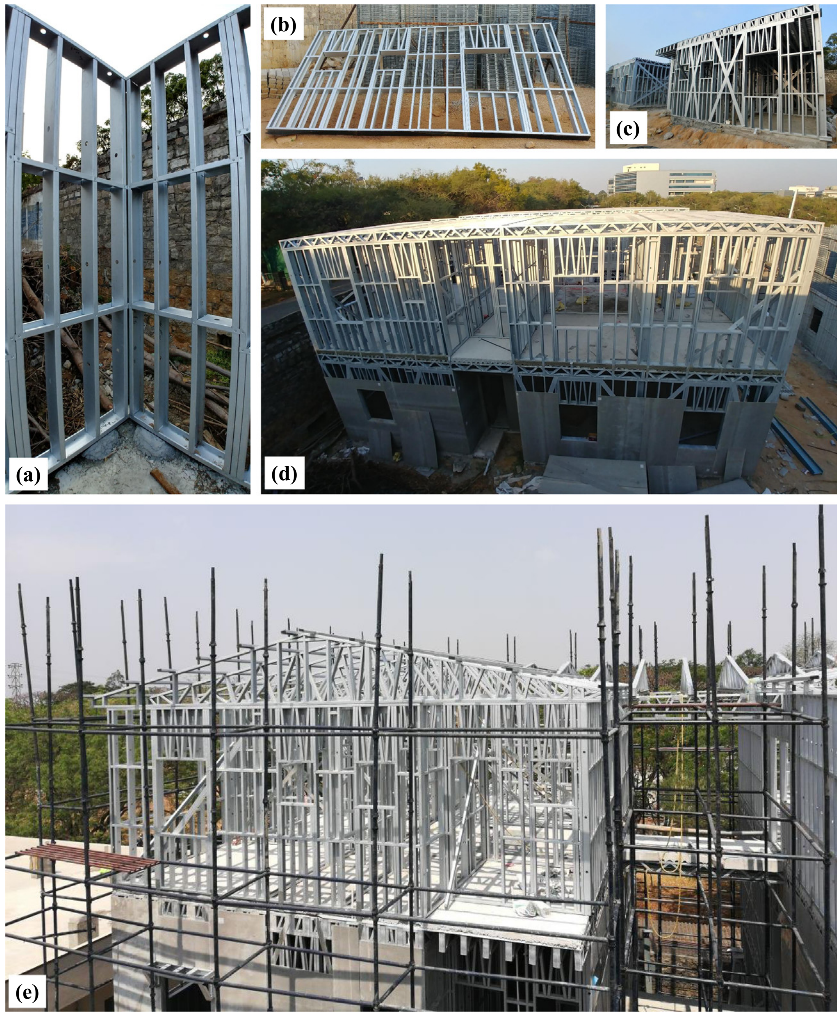

Fig. 3. View of the CFS Building Construction.

(gypsum and particle cement board) exhibits brittle and catastrophic failures against the twist of the C shaped CFS wall frame stud [20,21]. In addition, the design of sheathing braced CFS wall frame studs should consider several parameters such as slenderness [local (λ_l), distortional (λ_d) and global (λ_e)], cross-sectional properties and shape of the CFS wall frame stud, material composition, material properties, thickness of the sheathing board, sheathing configuration (single sided, double sided identical and non-identical), spacing between sheathing bracing connections and finally various screw types. However, the influence of these parameters are unknown for different loading cases that are typically encountered in CFS wall panel design. These impediments present significant challenges in the development of a generalized/robust design method for sheathing braced design concepts.

The currently available design guidelines for bracing design of CFS framing member are implicit (based on the design strength of the CFS wall frame stud alone) and do not consider the failure mode of various shapes of the CFS wall frame studs. In particular, the out-of-plane bending causes lateral torsional buckling and bi-axial bending for slender C (singly-symmetric) and Z (point-symmetric) shaped CFS wall frame studs, respectively; the same will undergo minor axis buckling and flexural-torsional buckling, respectively when subjected to axial compression. These buckling mode causes different forces at the screw joints/connections where the bracings are connected, i.e. the minor axis buckling and lateral torsional buckling causes shear force (Fig. 2f) and diagonal pulling force (Fig. 2h) at the screw/fastener which connects the CFS wall stud and bracing member, respectively. As per authors' knowledge the behaviour of bracing screw connections subjected to torsional forces have not been explored previously. The pioneer investigation on bracing systems by Winter [37] suggest to reduce the unbraced length of the CFS wall frame stud to take into account the effect of sheathing bracing connection in the design strength. Moreover, the AISI S210 [2] suggest to provide an additional steel bracing when

Table 1
Material properties of gypsum board.

Thickness of gypsum board (mm)	Tensile modulus E_g (MPa)	Ultimate strength F_r (N)
12.5 (1/2 in.)	2100 (mean)	387
15.0 (5/8 in.)	2272.1 (mean)	420

E_g - Tensile modulus of gypsum board; F_r - strength of gypsum board at rupture.

Table 2
Material properties of CFS sheet.

CFS sheet thickness (mm)	E_s (GPa)	f_y (MPa)	f_u (MPa)	ε_f (%)
1.5	210.93	377.36	440.67	17.53
2.5	215.87	329.9	419.83	18

E_s - Young's modulus of steel; f_y - yield strength of steel; f_u - ultimate tensile strength; εf - strain at fracture;

Table 3
Cross sectional dimensions and sheathing configurations of the hat shaped CFS wall frame stud test specimens.

Specimen ID[a]	Depth of web (h_w)	Breadth of flange (b)	Breadth of brim (l_h)	Depth of Lip (l_v)	Sheet thickness (t)	CFS wall frame stud slenderness (unsheathed)	
						Local (λ_l)	Global (λ_e)
H 01-12.5-150 H 01-12.5-300 H 01–15.0-150 H 01–15.0-300	50	50	22.5	–	1.5	0.69	2.34
HL 01–12.5-150 HL 01–12.5-300 HL 01–15.0-150 HL 01–15.0-300	50	40	15	15	1.5	0.30	2.02
H 02–12.5-150 H 02–12.5-300 H 02–15.0-150 H 02–15.0-300	95	50	20	–	1.5	0.70	2.70
HL 02–12.5-150 HL 02–12.5-300	95	52.5	15	15	1.5	0.48	2.51
H 03–12.5-150 H 03–12.5-300 H 03–15.0-150 H 03–15.0-300	50	50	50	–	2.5	0.63	1.13
HL 03–12.5-150 HL 03–12.5-300 HL 03–15.0-150 HL 03–15.0-300	50	47.5	15	28.5	2.5	0.19	1.30

[a] Labeling of specimens: Specimen ID – Sheathing board thickness- Sheathing bracing spacing; Graphical definition of h_w, b, l_h, l_v, t, are given in Fig. 2a; $\lambda_l = \sqrt{f_y/F_{crl}}$; $\lambda_d = \sqrt{f_y/F_{crl}}$; $\lambda_e = \sqrt{f_y/F_{cre}}$

Table 4
Cross sectional dimensions and sheathing configurations of the C shaped CFS wall frame stud test specimens.a

Specimen IDa	Depth of web (h_w)	Breadth of flange (b)	Depth of Lip (l_v)	Sheet thickness (t)	CFS wall frame stud slenderness (unsheathed)		
					Local (λ_l)	Distortional (λ_d)	Global (λ_e)
CL01–12.5-150 CL01–12.5-300 CL01–15.0-150 CL01–15.0-300	67.5	38	20	1.5	0.52	0.53	1.33
C01–12.5-150 C01–12.5-300 C01–15.0-150 C01–15.0-300	50	36	–	2.5	0.57	–	1.30
C02–12.5-150 C02–12.5-300 C02–15.0-150 C02–15.0-300	50	45	–	1.5	1.31	–	1.52
C03–12.5-150 C03–12.5-300 C03–15.0-150 C03–15.0-300	80	50	–	1.5	1.49	–	1.54
CL02–12.5-150 CL02–12.5-300 CL02–15.0-150 CL02–15.0-300	80	28.5	28.5	2.5	0.32	0.37	1.21
CL03–12.5-150 CL03–12.5-300 CL03–15.0-150 CL03–15.0-300	80	43.5	10	1.5	0.59	0.78	1.44
C04–12.5-150 C04–12.5-300 C04–15.0-150 C04–15.0-300	80	50	–	2.5	0.81	–	1.25
C05–12.5-150 C05–12.5-300 C05–15.0-150	50	65	–	1.5	1.87	–	1.25

[a] Labeling of specimens: Specimen ID – Sheathing board thickness- Sheathing bracing spacing; Graphical definition of h_w, b, l_v, t, are given in Fig. 2b; $\lambda_l = \sqrt{f_y/F_{crl}}$; $\lambda_d = \sqrt{f_y/F_{crl}}$; $\lambda_e = \sqrt{f_y/F_{cre.}}$

Table 5
Cross sectional dimensions and sheathing configurations of the point-symmetric CFS wall frame stud test specimens.

Specimen ID[a]	Depth of web (h_w)	Breadth of flange (b)	Depth of Lip (l_v)	Sheet thickness (t)	CFS wall frame stud slenderness (unsheathed)		
					Local (λ_l)	Distortional (λ_d)	Global (λ_e)
Z02–12.5-150 Z02–12.5-300 Z02–15.0-150 Z02–15.0-300	50	35	–	1.5	1.01	–	2.18
Z01–12.5-150 Z01–12.5-300 Z01–15.0-150 Z01–15.0-300	50	37.5	–	2.5	0.59	–	1.62
ZL02–12.5-150 ZL02–12.5-300 ZL02–15.0-150 ZL02–15.0-300	50	28.2	27.5	1.5	0.36	0.39	1.47
ZL01–12.5-150 ZL01–12.5-300 ZL01–15.0-150 ZL01–15.0-300	50	30	22.5	2.5	0.21	0.27	1.28
Z04–12.5-150 Z04–12.5-300 Z04–15.0-150 Z04–15.0-300	80	30	–	1.5	0.90	–	2.40
Z03–12.5-150 Z03–12.5-300 Z03–15.0-150 Z03–15.0-300	80	50	–	2.5	0.81	–	1.47
ZL04–12.5-150 ZL04–12.5-300 ZL04–15.0-150 ZL04–15.0-300	80	25	10	1.5	0.42	0.54	2.33
ZL03–12.5-150 ZL03–12.5-300 ZL03–15.0-150 ZL03–15.0-300	80	32	22.5	2.5	0.28	0.36	1.41

[a] Labeling of specimens: Specimen ID – Sheathing board thickness- Sheathing bracing spacing; Graphical definition of h_w, b, l_v, t, are given in Fig. 2c; $\lambda_l = \sqrt{f_y/F_{crl}}$; $\lambda_d = \sqrt{f_y/F_{crl}}$; $\lambda_e = \sqrt{f_y/F_{cre}}$

the length of the member exceeds 8 ft (2440 mm) (Fig. 1a). This length based design criterion by Winter [37] and AISI (S210) for sheathing bracing design may not be accurate because the cross-sectional properties of the structural member plays a vital role in the stability along with the length (L/r_y). Therefore, the length based bracing design recommendation may lead to inaccurate design and instability failures.

Contradicting to Winter [37] and AISI (S210), the latest AISI design specification S100 Section C2.2 [3] insists to avoid additional steel bracing when both flanges of the CFS wall frame studs are connected by sheathing board through the following statement: "*Where both flanges are so connected, no further bracing is required*". More importantly, this suggestion is independent of the length of the member, sheathing board, CFS wall frame stud type (C or Zed or Hat) and screw type. Further, the Eurocode (EN [11]) suggests the use of continuous profiled steel deck to restrain (brace) the instability failures of the all CFS structural members including Z, Hat, U, C and sigma ($\sum$) sections. Similar to EN [11], the AISI (S210) also recommends a steel deck with minimum of 15 mm rib depth or 9.5 mm wood sheathing board for bracing the CFS floor joists. Again, this recommendation by EN [11] and AISI (S210) is not accurate as the bracing strength and stiffness requirement depend on the design strength and failure mode of the CFS wall frame stud [40], and may lead to over conservative design. Hence, the concept of sheathing based bracing design shall be formulated with robust design parameters that considers the failure modes of CFS structural members, connection behaviour and in addition, the design shall adopt a demand and supply approach where the sheathing configuration (sheathing board material properties and spacing between the sheathing bracing connections) can be designed according to the need (failure modes and design strength of the CFS wall frame stud). However, at present, it is only possible to suggest a limitation for the use of the above

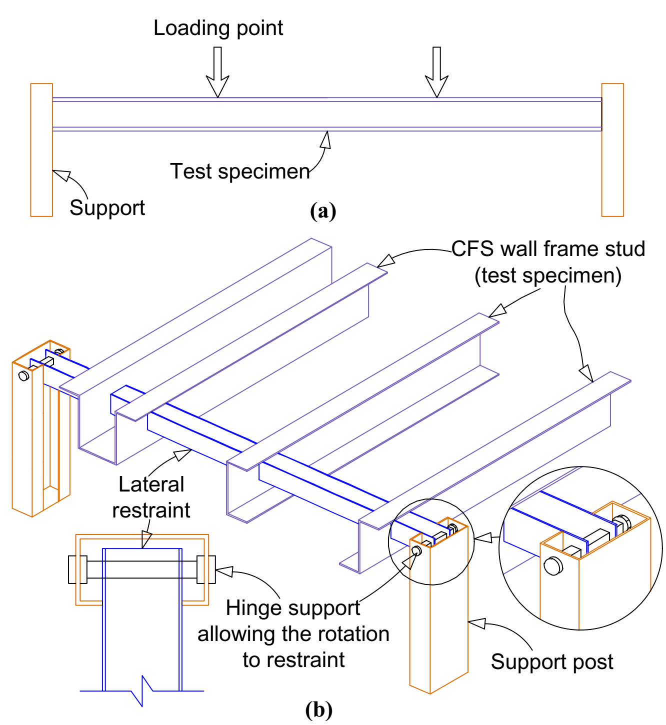

Fig. 4. Test setup proposed by AISI [6]: (a) Elevation view of the test setup; (b) Detailed view of the support conditions.

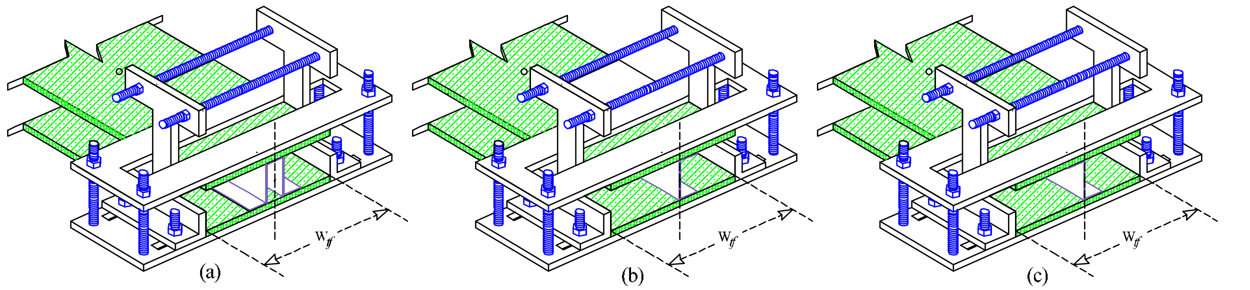

Fig. 5. Development of new test fixture for sheathed CFS wall panels subjected to out-of-plane loading: (a-f) View of the various shapes of the sheathed CFS wall frame studs with end fixture.

mentioned design specifications since the influence of most critical design parameters are unknown.

After having a good understanding of the current guidelines on the design of bracing systems for CFS wall frames, it is necessary to choose the appropriate and effective bracing systems among the two (conventional steel type or sheathing type), before beginning to investigate about the structural behaviour. As described previously, the inherent blemishes in the conventional steel bracings cannot provide an effective bracing system for CFS wall frames. In addition, Yura [40] concluded that the bracing that does not connect the flanges of the structural member will be inefficient as the flanges are more vulnerable to twist. Hence, the sheathing board bracing system that is attached on both the flanges of the CFS wall frame studs (Fig. 2f) has an additional advantage of reducing the steel consumption by eliminating the steel bracings which is the focus of the present investigation.

The available literature on sheathed CFS wall frames are focused on the structural behaviour of CFS member (mostly C shape) under axial compression and combined axial-bending rather than investigating the failure modes of different CFS wall frame stud shapes and sheathing bracing connection (sheathing to CFS stud by fastener). The sheathing braced design methods of both AISI [2,5] and EN [11] employ the following concepts; (i) the sheathing stiffness braces the member from global instability failure (Fig. 2d), (ii) the reduced minor axis slenderness [with an influence of sheathing bracing, unbraced length (L) reduced to two times of the sheathing bracing connection spacing ($2d_f$) as per AISI [3] and effective length factor $K = 0.5$ as per EN [11]] of the CFS wall frame stud can be considered for calculating the design strength but not less than the slenderness for major axis buckling with an unbraced length of L. It is important to note that a majority of the currently available literature reported the structural behaviour of sheathed C shaped CFS sections only, except by Winter et al. [38], while the industry uses hat shaped (Fig. 2a) CFS studs for wall panels and Z shaped (Fig. 2c) CFS studs for floor joists. The advantage of using Z shaped CFS studs for floor joints is that the Z shapes can be overlapped for connections and the hat shaped CFS stud provides excellent resistance against the torsional buckling due to its symmetric nature. Therefore, there is a need for investigating the effect of sheathing bracing on the slender C shape, hat and Z shaped CFS studs under worst case loading and develop design guidelines. Hence, the objective of this investigation is to explore the structural behaviour of various shapes of slender CFS wall frame studs that are vulnerable to fail in torsional buckling braced with sheathing boards and deduce appropriate guidelines or suggest limitations for the existing specifications towards developing a robust design method for bracing design. Nevertheless, this preliminary study attempts to investigate the bracing effect of one of the traditionally used sheathing board (gypsum) with all other design parameters of CFS wall frame stud. The material properties of the gypsum board and CFS studs obtained from tensile tests are summarized in Tables 1 and 2 respectively. The sample preparation, test setup and procedure for determining the material properties of the gypsum board is explained in Selvaraj and Madhavan [22]

3. Objective of present investigation, specimen selection, loading pattern and new test fixtures

The objective of this investigation is to understand the structural behaviour of various shapes of sheathed CFS wall frame studs, quantify the structural contribution of gypsum sheathing board (increase

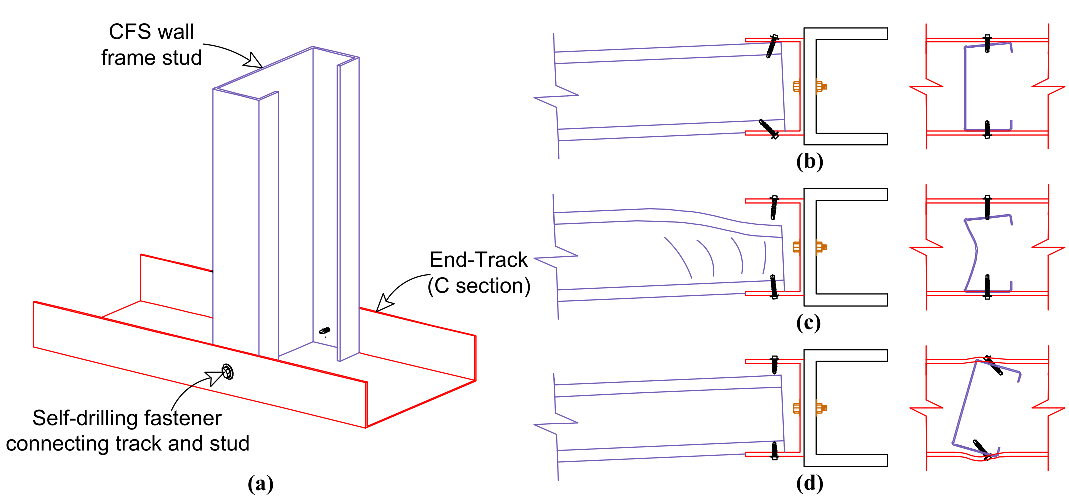

Fig. 6. (a) Typical end track connection of the CFS wall panel and corresponding failure modes due to out-of-plane loading; (b) Pull-out failure; (c) Web-crippling failure; (d) Twisting failure of the CFS stud.

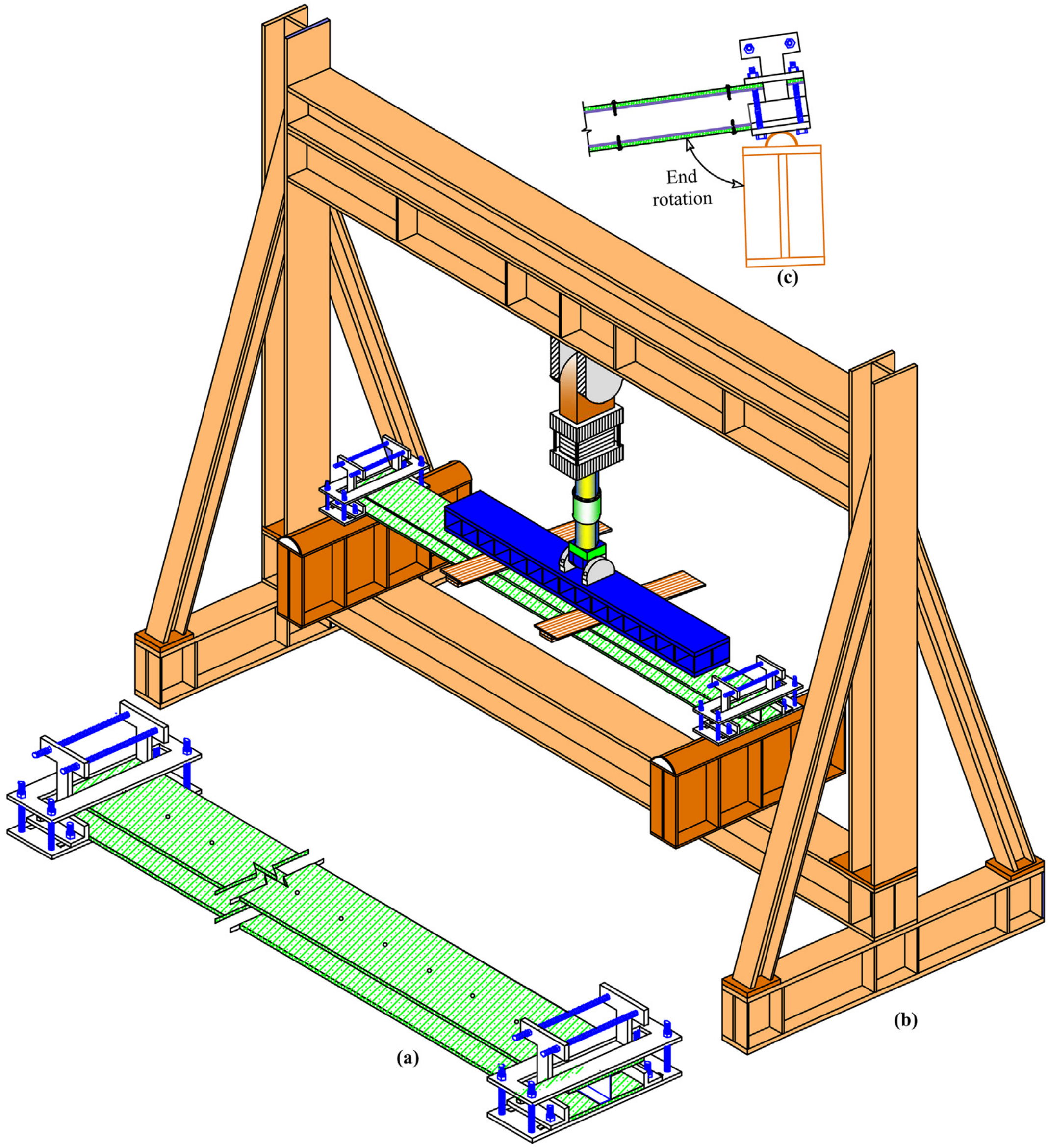

Fig. 7. Test setup for assessment of sheathing bracing connection effect on CFS stud failure modes: (a) Test specimen (sheathed CFS wall frame stud); (b) View of the loading frame; (c) Simulation of end rotations.

in the design strength of the member) and check the suitability of the AISI and Eurocode design methods (reduced slenderness concept). In addition, the recommendation of the Gypsum Association [13] for the maximum spacing between the fastener connections is verified. It is common that any bracing system is expected to completely brace the instability failure of the CFS wall frame stud. Thus the present investigation explores the ability of the gypsum sheathing bracing or in other words required sheathing bracing (sheathing configuration - sheathing thickness and spacing between the bracing connections) with respect to the slenderness or cross-sectional shape or design strength or generalized sectional property of the CFS wall frame stud is investigated. The study parameters include three different shapes of CFS wall frame studs (C shape - singly symmetric, Z shape - point symmetric and hat shape - doubly symmetric shown in Figs. 2a-2c), and in each CFS wall frame stud shapes the local (λ_l), distortional (λ_d), and global slenderness (λ_e) magnitudes are varied. Further, in each global slenderness (λ_e) the thickness of the gypsum sheathing board (t_b), and spacing between the sheathing bracing connections (d_f) are varied for exploring the structural behaviour of gypsum sheathing boards, and finally a total of 107 (Tables 3-5) number of specimens were tested. It should be noted that all the test samples are plain CFS members without web perforations since the objective of this investigation is to explore the structural contribution of gypsum sheathing boards in lieu of steel bracing. The strength improvement due to the effect of sheathing bracing connection is quantified by comparing the sheathing braced ($M_{DSM\text{-}Br}$) and unsheathed design strength ($M_{DSM\text{-}UN}$).

The global slenderness magnitudes of the CFS wall frame studs are used to group the test specimens for simple identification as the previous studies indicate that the global stability is increased due to sheathing bracing connection effect. In total, there are 22 group specimens including all shapes of the CFS studs and each group has five specimens including one unsheathed CFS wall frame stud and four sheathed CFS studs with the different sheathing configurations. The sheathing configurations are varied by changing the gypsum sheathing board thickness (t_b) and spacing between the sheathing bracing connections (d_f) as shown in Tables 3-5. The test specimens are labelled by their cross-sectional shape and sheathing configuration (i.e. CL01–12.5-150, H 01–12.5-300 and Z01–12.5-150) as follows: Specimen Identification number-Sheathing board thickness (in mm)-Sheathing bracing

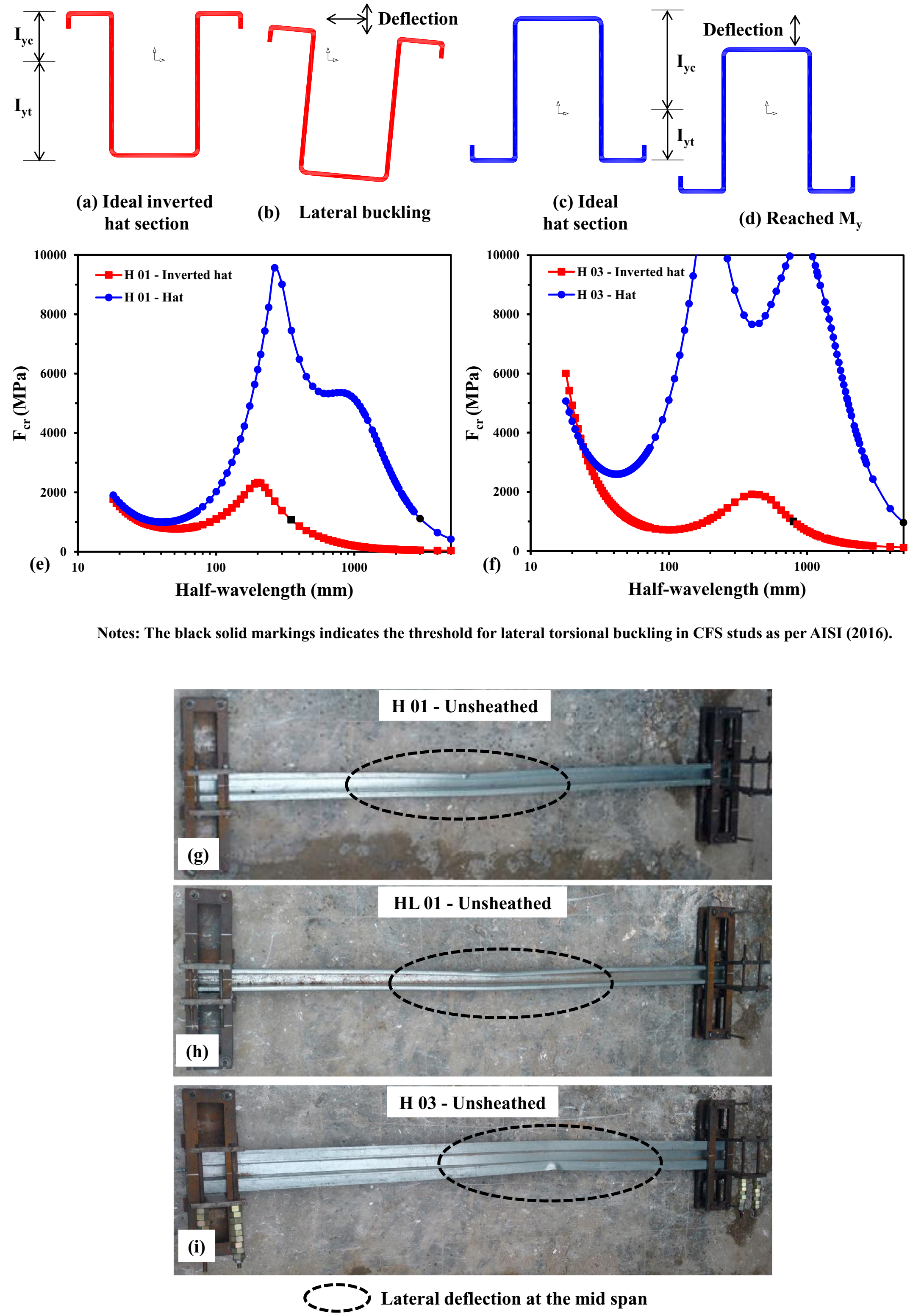

Fig. 8. (a-d) CFS hat sections and it corresponding failure modes; (e-f) Elastic buckling analysis of the hat and inverted hat shaped CFS studs; (g-i) Failure mode of the inverted CFS shaped unsheathed wall frame studs.

connection spacing (in mm). The failure mode of the various CFS cross-sections are obtained from unsheathed CFS wall frame stud tests and the structural contribution of the gypsum sheathing was determined by comparing the moment capacities of the unsheathed and sheathed CFS wall frame studs. All the test specimens (both sheathed and unsheathed) are of unbraced length 2250 mm and the out-to-out length is 2400 mm. The length of the test specimen is based on the AISI guidelines which require additional steel bracing should be provided when the length of the structural member exceeds or equal to 8 ft (2440 mm). However, for conservativeness the unbraced length of the test specimen is taken as 2250 mm. The width of the test specimen is 300 mm and the test specimens are of single CFS wall stud panels. It should

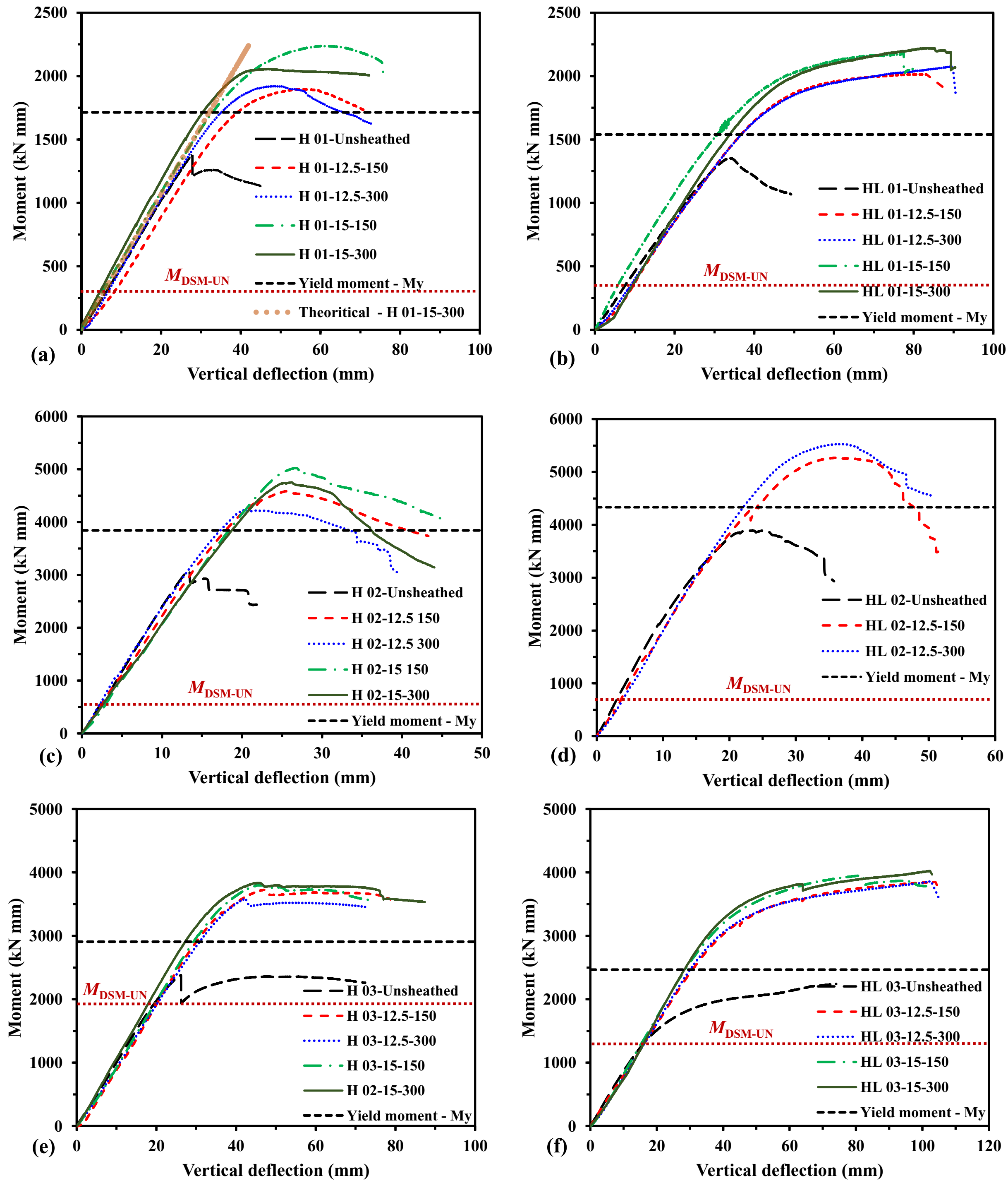

Fig. 9. Moment-deflection plots for the tested specimens – indicating the structural contribution of the gypsum sheathing.

be noted that the self-drilling fastener used in the present study is of No.6 type [8] and has a steel-cum rubber fastener suggested by Selvaraj and Madhavan [23].

4. Test setup for assessment of sheathing bracing connection effect on CFS stud failure modes

Having formulated the problem objective, it is chosen to apply out-of-plane loading and simulate simple support fixture conditional on the ends of sheathed CFS wall frame studs. This is because the previous studies on sheathed CFS structural members and bracings indicate that the occurrence of pull-through failures at the sheathing bracing connections is the governing mode of failure and arise due to torsional buckling (twist) of the CFS wall frame stud leading to a rift failure (separation) of the bracing member from CFS structural member [1]. The out-of-plane loading will cause lateral torsional buckling, biaxial bending and lateral buckling in C, Z and Hat-shaped CFS wall frame studs, respectively, which primarily induce torsional force at the sheathing bracing connections leading to a pull-through failures. However, the same member when subjected to axial compression loading may lead to minor axis

Table 6
Test results of the sheathed hat shaped CFS wall frame studs.

Specimen ID	CFS stud slenderness		Yield moment of the CFS stud (M_y) (kN mm)	Test results				M_{DSM-UN} (kN mm)	M_{DSM-Br} (kN mm)	M_{E-SH} / M_{DSM-Br}	M_{DSM-UN} vs. M_{DSM-Br}
				Unsheathed CFS Stud	Two-sided Sheathed CFS stud						
	λ_l	λ_e		M_{U-UN} (kN mm)	M_{E-SH} (kN mm)	Failure mode					
						CFS stud	Sheathing				
H 01–12.5-150	0.69	2.34	1714.39	1377.00	1896.63	LB in brim	No failure at the locations of	310.68	1714.39	1.11	451.8
H 01–12.5-300					1922.20	after	sheathing-fastener connections			1.12	
H 01–15.0-150					2239.72	yielding				1.31	
H 01–15.0-300					2056.70					1.20	
HL 01–12.5-150	0.30	2.02	1539.66	1352.90	2118.50	Yielding	Rupture crack in tension side	353.57	1539.66	1.31	335.5
HL 01–12.5-300					2021.27					1.31	
HL 01–15.0-150					2176.51					1.41	
HL 01–15.0-300					2222.87					1.44	
H 02–12.5-150	0.70	2.70	3843.50	3083.57	4590.81	LB in brim	No failure at the locations of	552.81	3843.50	1.19	595.3
H 02–12.5-300					4223.59	and web	sheathing-fastener connections but			1.10	
H 02–15.0-150					5021.02	after	breakage in the sheathing is due to LB			1.31	
H 02–15.0-300					4752.26	yielding				1.25	
HL 02–12.5-150	0.48	2.51	4332.56	3894.61	5270.06	Yielding		673.20	4332.36	1.22	543.5
HL 02–12.5-300					5528.25					1.28	
H 03–12.5-150	0.63	1.13	2907.82	2381.14	3726.96	LB in brim	No failure at the locations of	1949.62	2907.82	1.28	49.1
H 03–12.5-300					3587.72	after	sheathing-fastener connections but			1.23	
H 03–15.0-150					3802.12	yielding	breakage in the sheathing is due to LB			1.31	
H 03–15.0-300					3838.66					1.32	
HL 03–12.5-150	0.19	1.30	2467.39	2242.92	3850.14	Yielding		1318.50	2467.39	1.56	87.1
HL 03–12.5-300					3832.08					1.55	
HL 03–15.0-150					3952.85					1.60	
HL 03–15.0-300					4025.61					1.63	

M_{U-UN} - ultimate moment capacity of unsheathed CFS wall frame stud; M_y -Yield moment capacity of unsheathed CFS wall frame stud; M_{E-SH} - Ultimate moment capacity of the sheathed CFS wall frame stud; M_{DSM-UN}– Design moment capacity of the unsheathed CFS wall frame stud as per direct strength method (nominal); M_{DSM-Br}– Design moment capacity of the sheathed CFS wall frame stud as per direct strength method (nominal); LB- Local buckling;

buckling or local buckling and may not simulate the torsional forces at the sheathing bracing connections as in flexural loading.

The test setup used in this investigation is different from the recommended test fixture by AISI S908 [6] (Fig. 4) for determining the flexural strength of CFS structural members supported by roofing system (inherent bracing). The AISI suggests to use a lateral restraint at the ends of the specimens in the form of a steel member through the web perforations to avoid slip or twist at the ends as shown in Fig. 4b. There are few technical hitches in the test setup proposed by AISI [6]; (i) The perforations at the ends may result in web crippling failure; (ii) the lateral restraint at the ends will provide an additional bracing effect to the CFS member which will subdue the effect of wall sheathing or in other words, the realistic bracing contribution of the wall sheathing cannot be captured; (iii) the lateral restraint could fail before the failure of the CFS wall frame stud as the entire load is transferred through it. Therefore, to simulate the out-of-plane (major-axis) bending in the sheathed CFS wall frame studs and capture the true/actual bracing connection effect of the gypsum boards, a new support fixture has been developed as shown in Fig. 5.

In general, the CFS wall frame studs are typically connected with track connection at both the ends by self-drilling fasteners as shown in Fig. 1a. These end track connections may fail due to screw pullout (due to tension) or the CFS wall frame studs may twist due to insufficient end restraints due to out-of-plane loading or fail due to web crippling of CFS studs or exhibit track punch through failure as shown in Fig. 6 ([10,18] and Albert Victor [7]). When any of the above mentioned failure mode occurs, it is not possible to capture the true/actual gypsum sheathing bralcing connection effect, and hence a new support fixture is developed such that it avoids any unintentional concentrated loading and restrains the CFS stud ends from twisting [19,21,22]. The view of the various shapes of the sheathed CFS wall frame studs with end fixture is shown in Fig. 5. In addition, it should also be noted that the end fixtures are designed to allow out-of-plane rotations which simulates the simple support end conditions.

The testing program was carried out using MTS servo hydraulic actuator. The out-of-plane loading was applied in displacement control mode at a constant rate of 0.6 mm/min (Fig. 7). The loading was paused at three stages for 3 min each prior to achieving the ultimate loading capacity. This was done to ensure the application of static loading. The out-of-plane loading was applied through two loading points with an intermediate distance of 800 mm. The loading points are of 100 mm width (patch load) and has the corners chamfered to avoid any unintentional application of concentrated forces at the sheathing boards. The out-of-plane (vertical) and lateral displacements of the sheathed CFS wall frame studs were measured using non-contact displacement transducers (NCDT), but the recorded data indicated that the out-of-plane displacement measurement readings were affected due to the sudden occurrence of pull-through failure followed by separation of sheathing boards. Therefore, the applied out-of-plane displacement at the loading points were used for plotting the load vs. displacement graphs. The appropriateness of the test setup up was ensured by comparing the measured stiffness and theoretical stiffness of the sheathed CFS wall frame stud as shown in one of the moment versus deflection curve of the sheathed hat shaped CFS stud. The theoretical stiffness of the sheathed CFS wall frame stud is determined from the following simple mechanics expression $\triangle = Wb\ (3L^2 - 4b^2)/24EI_{composite}$, where $\triangle$is the midspan deflection; w is the one-half of the load applied; L is the unbraced length of the specimen; b is the intermediate distance between the support and loading point; $EI_{composite}$ is the composite flexural rigidity considering full-composite action between sheathing material and CFS stud [22].

5. Observations from sheathed CFS wall frame stud tests

5.1. General

Foremost, the failure modes exhibited by the unsheathed and sheathed CFS wall frame studs indicates that the new test fixture is

Table 7
Test results of the sheathed C shaped CFS wall frame studs specimens.

Specimen ID	CFS stud slenderness			Yield moment of the CFS stud (M_y) (kN mm)	Test results				$M_{DSM\ -UN}$ (kN mm)	$M_{DSM\text{-}Br}$ (kN mm)	$M_{E\text{-}SH}$ / $M_{DSM\text{-}Br}$	$M_{DSM\text{-}UN}$ vs. $M_{DSM\text{-}Br}$
					Unsheathed CFS Stud	Two-sided Sheathed CFS stud						
	λ_l	λ_d	λ_e		$M_{U\text{-}UN}$ (kN mm)	$M_{E\text{-}SH}$ (kN mm)	Failure mode: CFS stud	Failure mode: Sheathing				
CL01–12.5-150	0.52	0.53	1.33	2037.1	1379.7	1516.0	LTB	PT	1150.6	2037.1	0.74	–
CL01–12.5-300						1464.2	LTB	PT			0.72	–
CL01–15.0-150						2065.7	Y	NF			1.01	77.0
CL01–15.0-300						1856.8	LTB	PT			0.91	–
C01–12.5-150	0.57	–	1.30	1637.5	1341.8	1598.8	LTB	PT	966.8	1637.5	0.98	–
C01–12.5-300						1559.4	LTB	PT			0.95	–
C01–15.0-150						2225.1	Y	NF			1.36	69.4
C01–15.0-300						1838.9	Y	NF			1.12	69.4
C02–12.5-150	1.31	–	1.52	1412.0	1105.8	1221.9	LB	NF	560.0	1010.3	1.21	80.4
C02–12.5-300						1247.0	LB	NF			1.23	80.4
C02–15.0-150						1510.2	LB	NF			1.49	80.4
C02–15.0-300						1249.3	LB	NF			1.24	80.4
C03–12.5-150	1.49	–	1.54	2730.3	1467.3	2242.5	LB	NF	990.2	1794.7	1.25	81.2
C03–12.5-300						1945.8	LB	NF			1.08	81.2
C03–15.0-150						2566.9	LB	NF			1.43	81.2
C03–15.0-300						2459.5	LB	NF			1.37	81.2
CL02–12.5-150	0.32	0.37	1.21	3073.3	2400.3	2874.7	LTB	PT	2034.5	3073.3	0.94	–
CL02–12.5-300						2596.9	LTB	PT			0.84	–
CL02–15.0-150						3614.0	Y	NF			1.18	51.1
CL02–15.0-300						3046.7	LTB	PT			0.99	–
CL03–12.5-150	0.59	0.78	1.44	2632.6	1082.8	1827.3	LTB	PT	1268.7	2443.6	0.75	–
CL03–12.5-300						1858.9	LTB	PT			0.76	–
CL03–15.0-150						2865.1	D	NF			1.17	92.6
CL03–15.0-300						2500.6	D	PT			1.02	–
C04–12.5-150	0.81	–	1.25	3888	3400.8	3415.6	LTB	PT	2417.0	3773.7	0.91	–
C04–12.5-300						3452.5	LTB	PT			0.91	–
C04–15.0-150						3521.0	LTB	PT			0.93	–
C04–15.0-300						3308.1	LTB	PT			0.88	–
C05–12.5-150	1.87	–	1.25	1961.1	2478.0	2346.1	LB	NF	790.3	1098.7	2.14	39.0
C05–12.5-300						2362.2	LB	NF			2.15	39.0
C05–15.0-150						2645.9	LB	NF			2.40	39.0

$M_{U\text{-}UN}$ - ultimate moment capacity of unsheathed CFS wall frame stud; M_y -Yield moment capacity of unsheathed CFS wall frame stud; $M_{E\text{-}SH}$ - Ultimate moment capacity of the sheathed CFS wall frame stud; $M_{DSM\text{-}UN}$– Design moment capacity of the unsheathed CFS wall frame stud as per direct strength method (nominal); $M_{DSM\text{-}Br}$– Design moment capacity of the sheathed CFS wall frame stud as per direct strength method (nominal); LB - Local buckling; LTB – Lateral torsional buckling; D – Distortional buckling; Y – Yielding; PT – Pull-through; NF – No failure.

suitable for simulating the out-of-plane bending, as the experimental test setup exhibits the actual failure modes (Fig. 8g-8i) and the theoretical stiffness of the specimen is in sync with the experimentally obtained moment-deflection plots as shown in Fig. 9a. The overall observation indicates that the mechanics of CFS wall frame studs design is significantly influenced by failure mode of the sheathed CFS wall frame stud and sheathing-bracing connection. The test results indicate that the gypsum board sheathing on both flanges of the CFS wall frame stud is not suitable for resisting the lateral movement and rotation of all CFS studs, but it's applicability depends on the cross-section, slenderness, governing failure mode and design moment capacity of the CFS wall frame stud. The strength contribution of the gypsum board sheathing ranges from 39% to 595%, highly slender CFS studs has a significant gain in strength, however, this variation in strength also varies for different CFS stud cross-sections. Overall, the results shown in Tables 6-8 and Figs. 9-16 indicates that the current design recommendations of AISI S210 (Section B4), AISI S100 (Section C2.2) and EN [11] (Section 10.1) on sheathing bracing design requires modification. More detailed discussion on the bracing contribution of gypsum sheathing is explained in detail as follows:

6. Bracing contribution of gypsum sheathing for hat shaped CFS wall frame stud

Compared to the other conventional CFS stud shapes (C or Z), the hat shaped CFS stud has a distinct structural behaviour when loaded in major axis bending through positive and negative directions as it is unsymmetrical about the major axis (Fig. 8a and c). This is simply because the smaller I_{yc} (out-of-plane moment of inertia of the compression flange) leads to lateral buckling when the hat section is loaded negatively or reversed in position (Fig. 8a-b) (LTB) while the regular hat section (positive loading) does not fail in LTB as the I_{yc} is significantly higher than I_{yt} (Fig. 8c-d). This has been mechanically demonstrated in Fig. 8. The elastic buckling analysis (Fig. 8e-f) also shows that the regular hat section has the higher global buckling resistance compared to the reversely loaded hat section. More precisely, the elastic buckling analysis results indicate that the regular hat shaped CFS wall frame studs (Fig. 8e-f) does not fail in lateral buckling for a unbraced length up to 2650 mm as their global buckling stress is higher than 2.78 times of f_y (according to AISI [3] section C2.2), hence additional bracing is not required. While the reversely loaded hat shaped CFS wall frame studs (Fig. 8e-f) failed in lateral buckling ($f_{cr} < 2.78f_y$) at the unbraced length range of 300–800 mm, necessitating the additional bracing. Considering this theoretical evaluation, the CFS hat sections were tested in reversed position as shown in Fig. 5a.

A total of 6 different hat shaped CFS frame stud cross-section were chosen with a global slenderness (λ_e) ranging from 1.13 to 2.7 to investigate the structural effect of gypsum sheathing bracing connections. The parameters such as thickness of gypsum sheathing board (t_b) and spacing between the sheathing bracing connections (d_f) were also investigated, totalling to 30 number of specimens. The dimensions, structural properties and the failure mode of the reversely loaded hat shaped CFS studs are summarized in Table 3.

Table 8
Test results of the sheathed point symmetric CFS wall frame studs.

Specimen ID	CFS stud slenderness			Yield moment of the CFS stud (M_y) (kN mm)	Test results				M_{DSM-UN} (kN mm)	M_{DSM-Br} (kN mm)	M_{E-SH} / M_{DSM-Br}	M_{DSM-UN} vs. M_{DSM-Br}
					Unsheathed CFS Stud	Two-sided Sheathed CFS stud						
	λ_l	λ_d	λ_e		M_{U-UN} (kN mm)	M_{E-SH} (kN mm)	Failure mode: CFS stud	Failure mode: Sheathing				
Z02–12.5-150	1.01	–	2.18	1137.5	439.4	1029.95	Y*	PT	232.59	943.9	1.09	305.8
Z02–12.5-300						625.70	LTB	PT			0.66	–
Z02–15.0-150						1309.62	Y*	NF			1.39	305.8
Z02–15.0-300						1220.97	Y*	NF			1.29	305.8
Z01–12.5-150	0.59	–	1.62	1696.3	1074.7	907.03	LTB	PT	618.50	1696.3	0.53	–
Z01–12.5-300						853.34	LTB	PT			0.50	–
Z01–15.0-150						1520.36	LTB	PT			0.90	–
Z01–15.0-300						1425.32	LTB	PT			0.84	–
ZL02–12.5-150	0.36	0.39	1.47	1121.1	549.9	1134.26	Y*	NF	503.03	1121.1	1.01	122.9
ZL02–12.5-300						1142.41	Y*	NF			1.02	122.9
ZL02–15.0-150						1399.47	Y*	NF			1.25	122.9
ZL02–15.0-300						1187.08	Y*	NF			1.06	122.9
ZL01–12.5-150	0.21	0.27	1.28	1637.1	1122.4	1409.43	LTB	PT	945.19	1637.1	0.86	–
ZL01–12.5-300						1080.20	LTB	PT			0.66	–
ZL01–15.0-150						1655.24	Y*	NF			1.01	73.2
ZL01–15.0-300						1517.48	LTB	NF			0.93	–
Z04–12.5-150	0.90	–	2.40	1841.5	632.8	1948.61	LB	NF	313.95	1656.7	1.18	427.7
Z04–12.5-300						1715.92	LB	NF			1.04	427.7
Z04–15.0-150						2192.14	LB	NF			1.32	427.7
Z04–15.0-300						2072.28	LB	NF			1.25	427.7
Z03–12.5-150	0.81	–	1.47	3888.0	1819.4	1730.01	LTB	PT	1743.36	3757.97	0.46	–
Z03–12.5-300						1685.30	LTB	PT			0.45	–
Z03–15.0-150						2288.56	LTB	PT			0.61	–
Z03–15.0-300						2369.31	LTB	PT			0.63	–
ZL04–12.5-150	0.42	0.54	2.33	1857.9	339.0	1583.51	LTB	PT	337.14	1857.9	0.85	–
ZL04–12.5-300						1184.55	LTB	PT			0.64	–
ZL04–15.0-150						1892.21	Y*	NF			1.02	451.1
ZL04–15.0-300						2107.88	Y*	NF			1.13	451.1
ZL03–12.5-150	0.28	0.36	1.41	3364.0	1499.0	1635.07	LTB	PT	1638.98	3364.0	0.49	–
ZL03–12.5-300						1830.42	LTB	PT			0.54	–
ZL03–15.0-150						2298.66	LTB	PT			0.68	–
ZL03–15.0-300						2800.15	LTB	PT			0.83	–

M_{U-UN} - ultimate moment capacity of unsheathed CFS wall frame stud; M_y -Yield moment capacity of unsheathed CFS wall frame stud; M_{E-SH} - Ultimate moment capacity of the sheathed CFS wall frame stud; M_{DSM-UN}– Design moment capacity of the unsheathed CFS wall frame stud as per direct strength method (nominal); M_{DSM-Br}– Design moment capacity of the sheathed CFS wall frame stud as per direct strength method (nominal); LB - Local buckling; LTB – Lateral torsional buckling; D – Distortional buckling; Y* – Yielding; PT – Pull-through; NF – No failure.

As expected, the failure mode of the unsheathed reversely loaded hat shaped CFS wall frame studs is lateral buckling with higher lateral displacement (Figs. 8g-i) and the moment capacity is less than the corresponding yield moment capacity (M_y) (Figs. 9a-f - see moment-deflection curves with legend "Unsheathed"). This lateral buckling was completely hindered by the gypsum sheathing bracing connection effect in all the reversely loaded hat shaped CFS studs and led to reach the corresponding yield moment capacity (M_y) as shown in

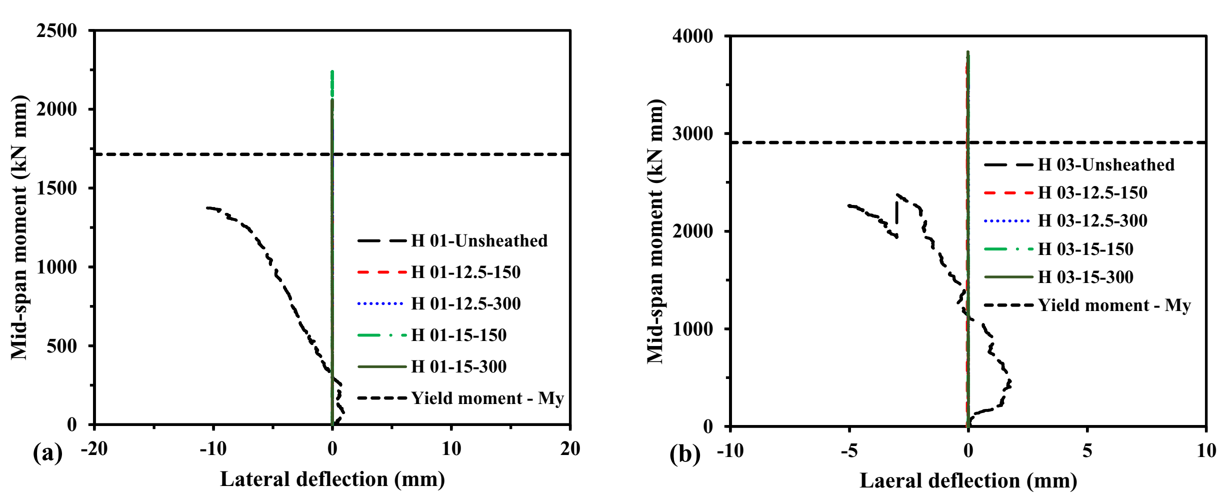

Fig. 10. Moment-lateral deflection plots for the H01 and H03 set specimens – indicating the lateral stability provided by the gypsum sheathing.

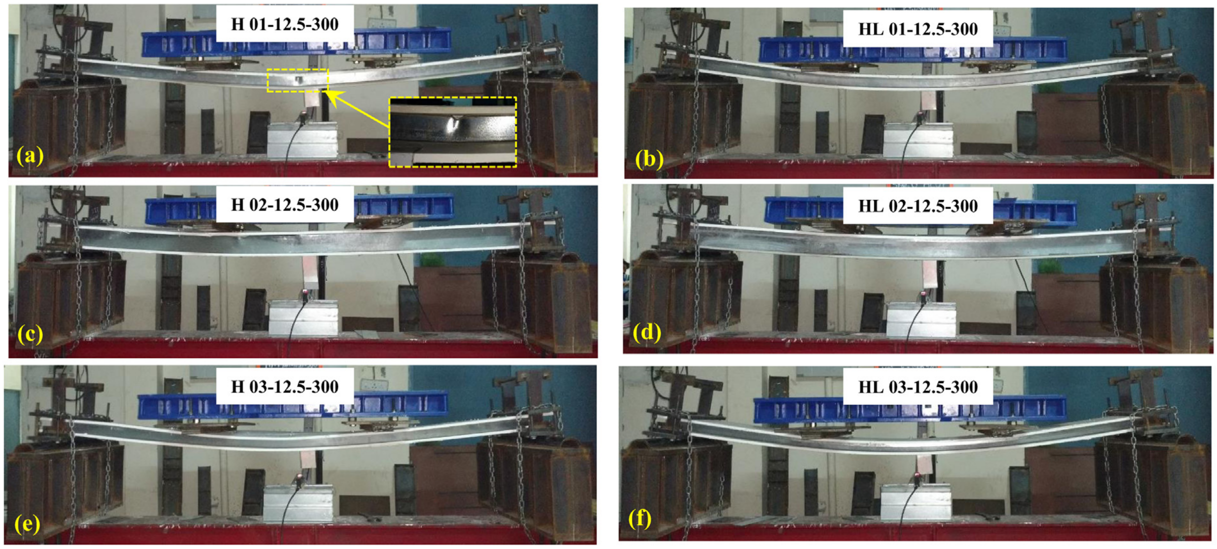

Fig. 11. Failure mode of the sheathed inverted CFS hat sections.

Figs. 9a-f. To further ensure the lateral stability provided by the gypsum sheathing, the lateral displacement obtained from the unsheathed and sheathed CFS wall frame studs were compared. The comparison indicates that the lateral displacement of sheathed CFS wall frames are negligible in all the specimens as shown in Fig. 10a-b. In addition, no significant damage or failure was observed at the sheathing bracing connections of reversely loaded sheathed CFS hat sections (Fig. 11) even though the gypsum board is a soft material and less tensile modulus (Table 1). This inhibition of lateral buckling may be due to the following facts: (i) the sheathing board on top flange behaves like a lean-on bracing (continuous) with regular spacing, consequently it arrests the lateral movement of the CFS member (ii) the reversed CFS hat section is structurally transformed to a closed cross section with a sheathing on top of it, thus it behaves like a tubular cross-section which has a higher torsional rigidity [36]. This insignificant difference between the failure modes of the sheathing bracing connections with respect to the sheathing board configuration (t_b and d_f) indicate that the 12.5 mm thick sheathing and 300 mm bracing connection spacing is sufficient to inhibit the lateral buckling. However, considering the possibility of over driven and dampness effect of gypsum sheathing, it is recommended to use the 15 mm thick sheathing with 150 mm bracing spacing. Therefore, it can be concluded that the recommendation of AISI S100 Section C2.2 [3] to eliminate the additional bracing when the CFS studs is sheathed on both the sides is suitable for sheathing braced design of CFS wall frame hat sections. The suggestion of Eurocode (EN [11]) to provide continuous profiled steel deck to hinder the instability failure of the CFS studs, however may be modified for effective design.

Though it is proven that the gypsum sheathing is sufficient to brace the reversely loaded hat shaped CFS stud from lateral buckling, it is necessary to have a design method to predict the design strength of the sheathing braced CFS wall frame stud. The Direct Strength Method (-Section F) of AISI [3] is used in this investigation to determine the design moment capacity as there is a possibility of occurrence of local (Fig. 11a) or distortional buckling in the CFS studs. The value of elastic critical lateral torsional buckling stress (f_{cre}) in DSM method is taken as 2.78 times of yield stress (f_y) as the CFS is fully braced. More detailed discussion about the DSM design rules for the sheathed CFS walls may be obtained from Selvaraj and Madhavan [19,20]. The design result shows that the influence of sheathing bracing connections increased ($M_{DSM\text{-}UN}$ vs. $M_{DSM\text{-}Br}$) the flexural moment capacity of the CFS wall frame studs by 49% to 595% compare to their corresponding unsheathed design moment capacity ($M_{DSM\text{-}UN}$) as shown in Table 6. This experimental investigation indicates that the consideration of structural contribution of inherent sheathing board increases the design strength of structural member and eliminates the need of additional steel bracing up to 2.44 m of unbraced length.

7. Bracing contribution of gypsum sheathing for C shaped CFS wall frame stud

The CFS channel section (C shaped) is prone to fail in lateral torsional buckling due to its low torsional resistance and singly symmetric shape when subjected to out-of-plane loading. A total of eight different CFS channels section were chosen based on the slenderness. The available literature on bracing design indicates that the CFS C channel section with high global ($\lambda_e > 1$) and with local ($\lambda_l > 1$) slenderness require less sheathing bracing connection effect to hinder LTB while the channel section with high global slenderness ($\lambda_e > 1$) and low local slenderness ($\lambda_l < 1$) requires sheathing bracing connection effect depends on the global stiffness of the member. Therefore, all the eight CFS cross sections were chosen to fail in lateral torsional buckling ($\lambda_e > 1$) and only three of them were designed to fail in local buckling in addition to LTB ($\lambda_e > 1$ and $\lambda_l > 1$) (Table 4) for verification of bracing requirement and sheathing bracing connection failure modes. A total of 39 specimens including unsheathed and sheathed CFS studs (two sided gypsum sheathing configuration) were tested and grouped into eight groups based on the global slenderness (λ_e) as shown in Table 4.

In general, the influence of sheathing bracing connection against the failure mode of the CFS C channels are in contrast to the failure mode of reversely loaded hat sections due to the explicit difference in failure modes of inversely loaded hat (lateral buckling - Figs. 8g-i and 11) and C channel sections (lateral torsional buckling - Figs. 12-13). The following are the observations from the experimental results of sheathed C channel sections:

(i) The lateral torsional buckling of C channels caused diagonal forces at the sheathing bracing connections and led to pull through failure of fasteners as shown in Fig. 14a.
(ii) The pull-through failure at sheathing bracing connections indicates that the gypsum board is able to withstand against the lateral movement (occurred in hat shaped stud as shown in

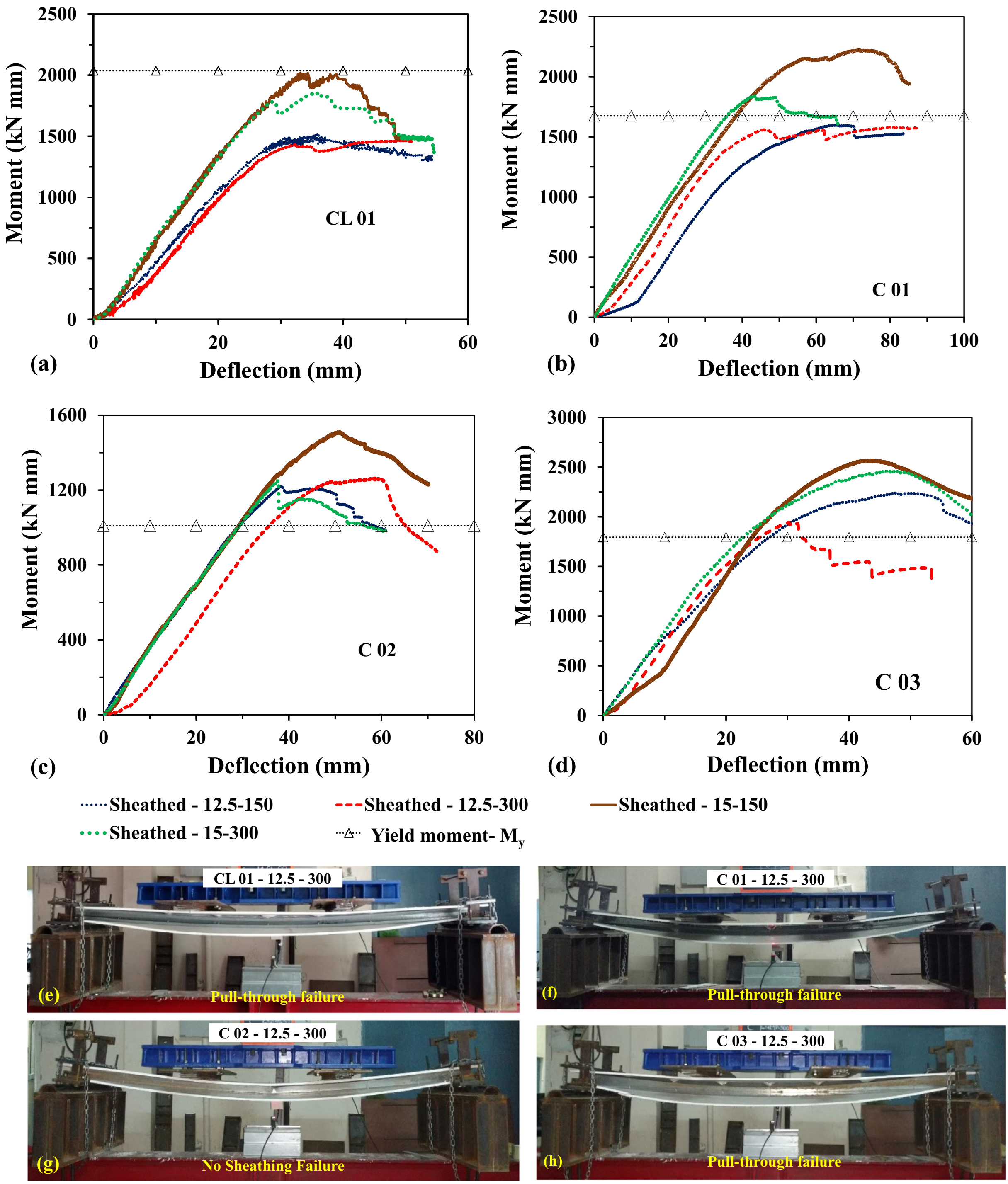

Fig. 12. Failure mode of the sheathed CFS C channel sections: (a-d) Moment-vertical deflection plots; (e-h) Failure modes.

Figs. 8g-i and 10-11) but capitulate against the cross-sectional twist of the C channel sections.

(iii) As expected, the influence of gypsum sheathing bracing connection vary significantly with respect to the global and local slenderness (governing failure mode) of the member. Further the sheathing configuration (thickness of the sheathing board (t_b) and spacing between the bracing connections (d_f)) also influences the sheathing bracing connection effect.

(iv) 12.5 mm thick gypsum sheathing board: The CFS studs with the vulnerability of both local and global buckling ($\lambda_e > 1$ and $\lambda_l > 1$) did not exhibit lateral torsional buckling and thereby reaching braced design moment capacity ($M_{DSM\text{-}Br}$) without any failure in the sheathing bracing connection locations as shown in Figs. 12c, g, 13d and h. However, 12.5 mm thick gypsum sheathing (with both 150 mm and 300 mm sheathing bracing connection spacing) was not sufficient to hinder the lateral torsional buckling of CFS C channels (Figs. 12a, b, d, e, f, h, 13a-c, and 13-e-13 g) that are vulnerable to fail only in global buckling ($\lambda_e > 1$ and $\lambda_l < 1$).

(v) This hindering effect of gypsum sheathing for CFS studs with higher global and local slenderness could be due to the fact that while sheathing braces the LTB ($\lambda_e > 1$), the next possible failure

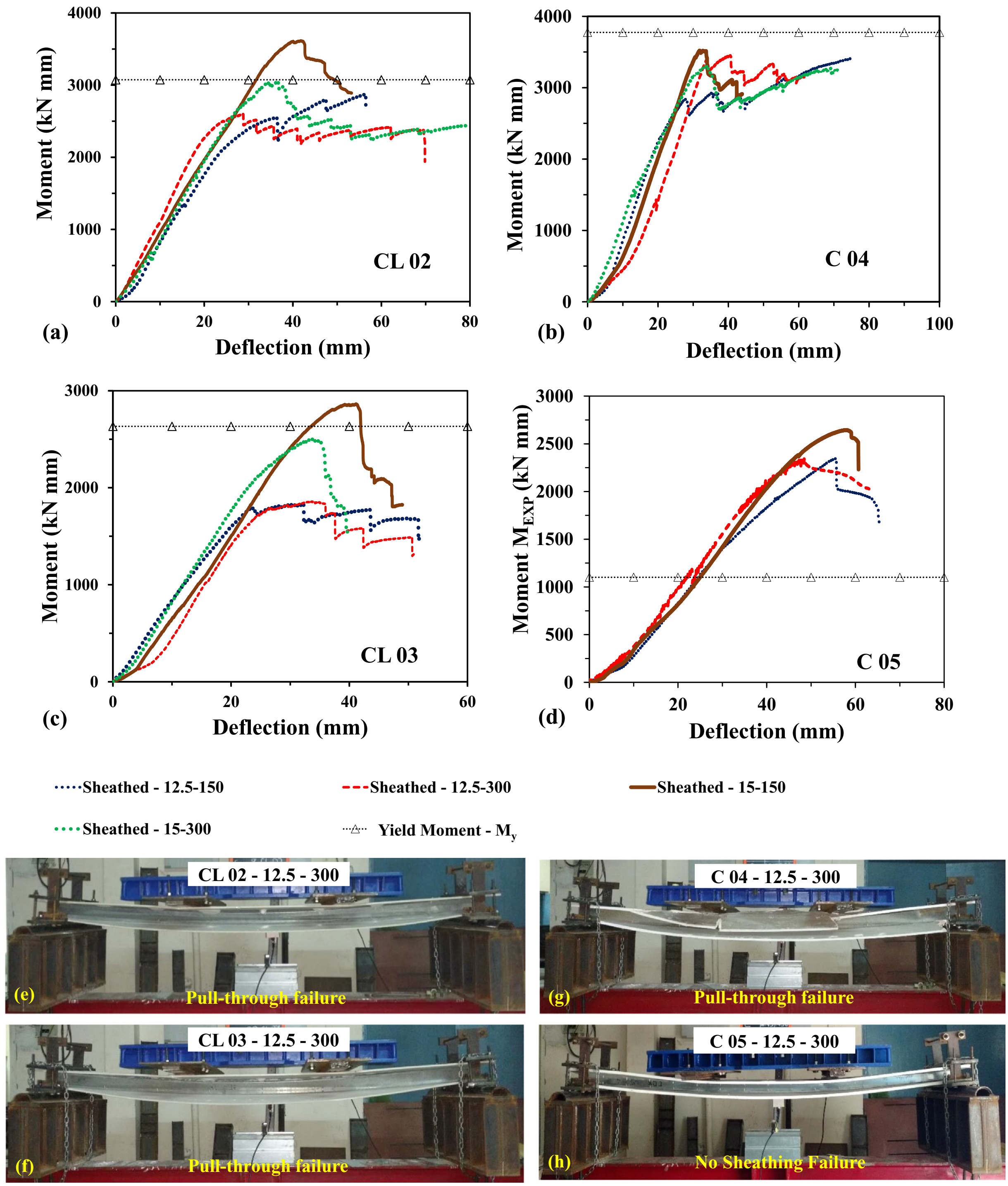

Fig. 13. Failure mode of the sheathed CFS C channel sections: (a-d) Moment-vertical deflection plots; (e-h) Failure modes.

mode which typically is local buckling ($\lambda_l > 1$) occurs in the CFS studs. Hence, any further demand for bracing becomes redundant.

(vi) 15 mm thick gypsum sheathing board: The closer sheathing bracing connection spacing 150 mm was effective in bracing all the tested CFS studs (see $M_{DSM\text{-}UN}$ vs. $M_{DSM\text{-}Br}$) except C04 set as shown in Table 7. This ineffective sheathing bracing connection effect for C04 set specimens creates an ambiguity on how the CFS studs with higher resistance against LTB is completely braced (λ_e of CL02 is 1.21), but the CFS studs with lesser resistance against LTB (λ_e of CL04 is 1.25) was not restrained. The above ambiguity necessitates more investigation on design parameters to be considered for sheathing braced design of CFS wall frame stud.

(vii) The influence of sheathing bracing connection increased the design strength ($M_{DSM\text{-}Br}$) of the CFS wall frame stud by 39%–81% (Table 7 – Column titled - $M_{DSM\ \text{-}UN}$ vs $M_{DSM\text{-}Br}$) compared to their corresponding unsheathed design strength ($M_{DSM\ \text{-}UN}$).

Although, the gypsum sheathing board was able to restrain the LTB failure of the C channels with higher global and local slenderness, no specific recommendation for the use of gypsum sheathing in CFS wall

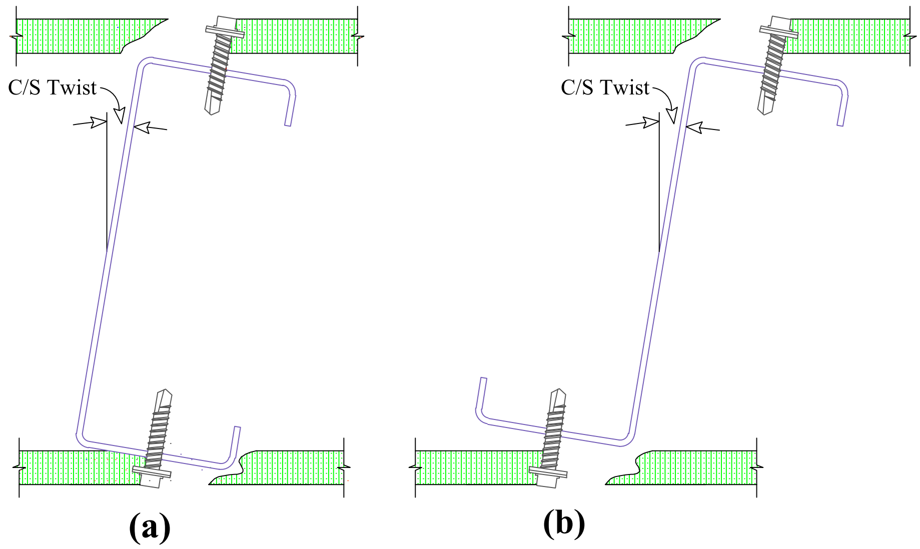

Fig. 14. Failure mode of the sheathed CFS Z channel sections: (a-d) Moment-vertical deflection plots; (e-h) Failure modes.

frame stud was deduced based on the 39 test results as no trend was observed in failure modes and strength improvement with respect to global slenderness magnitude. However, it is proven from the test results that the specification of AISI (S100–2016 – Section C2.2) to eliminate the additional steel bracing when sheathing attachment is provided on both the sides of the CFS wall panels shall be modified with more specific details such as sheathing type, sheathing thickness and sheathing bracing connection spacing.

7.1. Bracing contribution of Gypsum Sheathing for Point-symmetric shaped CFS wall frame stud

The point symmetric shaped CFS wall frame studs subjected out-of-plane loading through the plane of web fail in significant cross-sectional twist rather than lateral displacement before yielding as the plane of web does not coincide with principal plane (Fig. 2c) [17]. Though this cross-sectional twist of point-symmetric CFS wall frame stud may cause a similar failure mode at the sheathing bracing connection compared to the C shaped CFS member (Fig. 14), the structural effect of gypsum sheathing bracing is unknown. Therefore, a total of eight sheathed CFS stud specimen sets were tested, totalling to 40 specimens. The details of the specimen dimensions, slenderness, and unsheathed design strengths are summarized in Table 5. The influence of sheathing bracing in point symmetric CFS studs is similar to C channels, as: (i) the improvement in strength increases with the increase in global slenderness, (ii) the larger sheathing connection spacing (300) mm is not sufficient to brace the member from cross-sectional twist, (iii) the point symmetric CFS studs with global slenderness 1.28 has the strength improvement of 73.2% (Table 8 - Column titled - $M_{DSM\text{-}UN}$ vs $M_{DSM\text{-}Br}$) which is proportional to the strength improvement of 77% for the global slenderness of 1.33 in CFS C channels (Table 7). The maximum design strength improvement is 451% and 427%, respectively for the global slenderness of 2.33 and 2.4 as shown in Table 8 and Figs. 15-16. However, of the 32 two-sided gypsum sheathed specimens tested, only 14 of them were braced from biaxial bending, in other words only 14 of them were reached its braced moment capacity ($M_{DSM\text{-}Br}$), indicating that the current AISI design specification (Section C2.2) which suggest to eliminate the additional steel bracing is not suitable for sheathing braced design of point-symmetric shaped CFS wall frame studs.

Therefore, further analysis is carried out using the 107 test results of this study and a new rules for sheathing braced design of CFS wall frame studs with gypsum sheathing subjected to out-of-plane loading is suggested.

7.2. Structural behaviour of gypsum sheathing against cross-sectional twist of the CFS stud

Although the comprehensive test results from the current investigation indicates that the sheathing braced design strength of CFS wall frame studs depend on the cross-sectional shape, global slenderness and sheathing configuration (t_b and d_f), no conclusive evidence was drawn from this study regarding the sheathing braced design of CFS wall frame stud. This is because the trend of effectiveness in sheathing bracing is not proportional as the CFS C channels with global slenderness 1.44 (CL03) and 1.21 (CL02) were braced from LTB while the CFS C channels with intermediate global slenderness 1.25 (CL04) is not braced. Nevertheless, the CFS C channel with global slenderness 1.25 and local slenderness 1.87 was braced from LTB, indicating that there is a need for having an interdependent design parameter which considers both the design strength and pull-through failure of the CFS stud in the sheathing braced design concept. Therefore, the design parameters used by Yura [40] for design of bracings for beams shall be investigated for the two-sided sheathed CFS wall frame studs. The Yura method of bracing design is a supply-demand based approach where the required strength of bracing is determined from the ratio of design moment capacity of the beam to the depth of the beam [40] and the strength of the bracing is determined by a simple stress approach (required bracing strength = yield stress x cross sectional area). However, it should be noted that the method to determine the strength of the sheathing bracing connection against the pull-through connection is unknown until the present study, therefore, it is only possible to suggest a limitation for the use of AISI's guidelines for sheathing braced design. Hence, the design parameters recommended by Yura [40] is applicable to set a limitation for the use of gypsum as bracing rather than using an unconservative suggestion of AISI (-Section C2.2–2016).

In the present study, the ratio to the braced design moment capacity ($M_{DSM\text{-}Br}$) and depth of the web (h) is directly compared against the $M_{E\text{-}SH}$ / $M_{DSM\text{-}Br}$ ratio to determine the limitation for the use of gypsum sheathing (Fig. 17). The limitation ratio ($M_{DSM\text{-}Br}$/h) is fixed when the $M_{E\text{-}SH}$ / $M_{DSM\text{-}Br}$ value is less than unity. It should be noted that the braced design strength ($M_{DSM\text{-}Br}$) is determined with the resistance

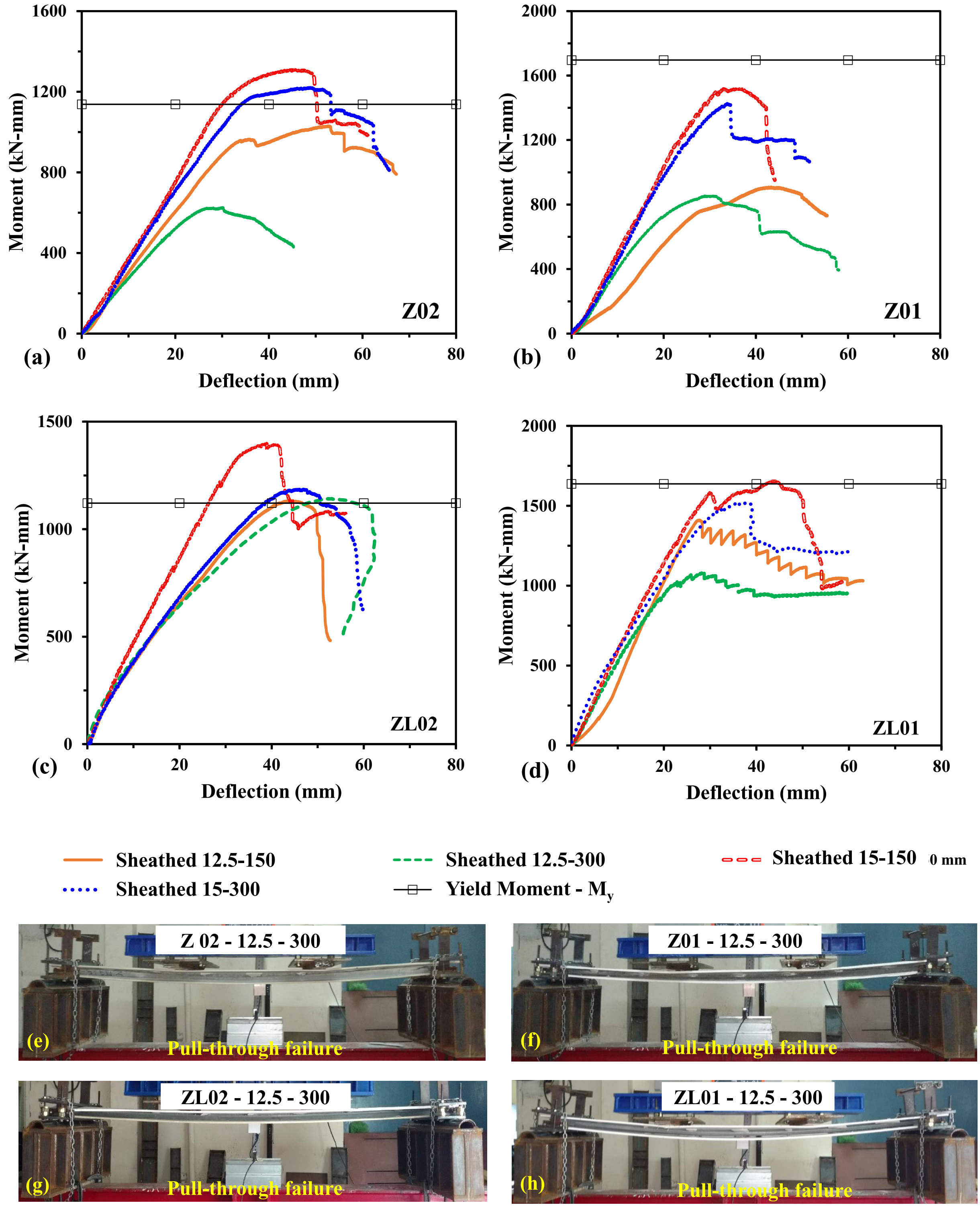

Fig. 15. Failure mode of the sheathed CFS Z channel sections: (a-d) Moment-vertical deflection plots; (e-h) Failure modes.

factor of 0.9. The comparison indicates that the 12.5 mm thick gypsum board with 300 mm sheathing bracing connection shall not be considered as a structural bracing as most of the $M_{E\text{-}SH}$ / $M_{DSM\text{-}Br}$ ratio is less than unity (Fig. 17). Therefore, the maximum sheathing bracing connection spacing limitation by GA [13] specification may be modified to 150 mm. Further, the gypsum sheathing boards of thickness 12.5 mm and 15.0 mm respectively with 150 mm and 300 mm sheathing bracing connection spacings may be considered as a structural bracing up to the $M_{DSM\text{-}Br}$/h ratio of 20 only as shown in Fig. 17. However, the gypsum sheathing of thickness 15 mm with 150 mm sheathing bracing connection spacing shall be used as a structural bracing in CFS wall frame studs up to a maximum $M_{DSM\text{-}Br}$/h ratio of 28.

7.3. Design implications for the sheathing braced slender CFS wall frame studs

Based on the results, the following specific conclusions can be drawn and may be adopted in the current AISI ([4] and S100–2016) and

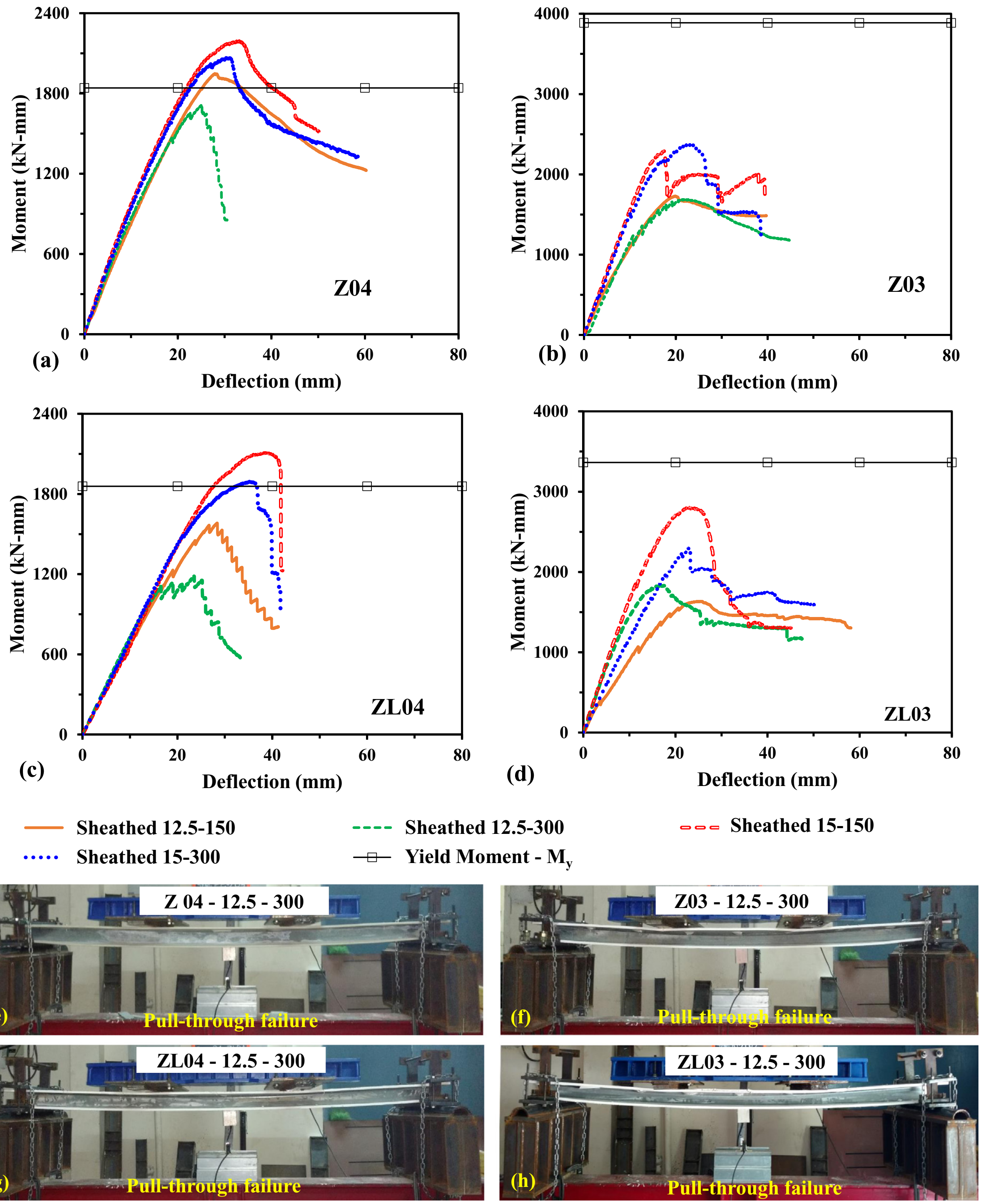

Fig. 16. Pull-through failure at the sheathing bracing connections caused by cross-sectional twist of the CFS stud.

Eurocode (EN [11]) design specification for the sheathing braced design:

1. The hat or other symmetric (against the axis of loading) shaped CFS wall frame studs needing lateral bracing may be attached to 15 mm thick gypsum sheathing with 150 mm sheathing bracing connection spacing.
2. The singly or point symmetric shaped CFS wall frame studs with a $M_{DSM\text{-}Br}/h$ ratio of 20 or less may be attached to 12.5 mm and 15 mm thick gypsum sheathing respectively with 150 mm and 300 mm sheathing bracing connection spacing, respectively for lateral bracing. For $M_{DSM\text{-}Br}/h$ ratios higher than 20 but less than 28, the gypsum bracing sheathing should be of 15 mm thick with 150 mm sheathing bracing connection spacing.

The suggested limits are applicable only for the gypsum sheathing connection by a fastener with steel cum rubber washer. It is expected that the suggested limits ($M_{DSM\text{-}Br}/h$) for the gypsum sheathing bracing may be reduced when the fastener without washers are used, due to less resistance.

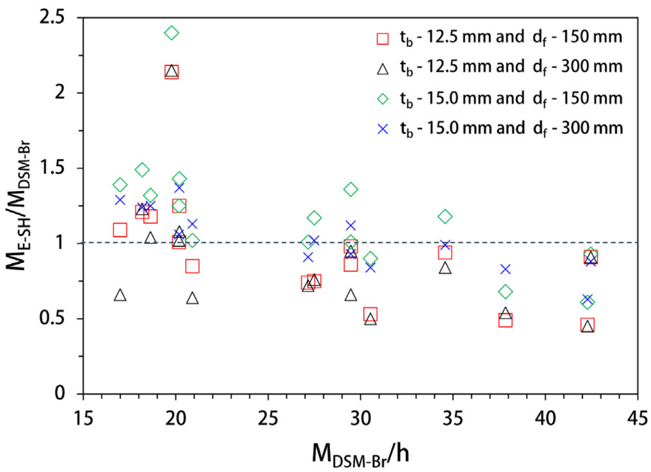

Fig. 17. Limitation of the gypsum sheathing for structural bracing.

8. Conclusions

This paper presents a series of experimental results to investigate the structural behaviour of CFS wall frame stud with gypsum sheathing, in which the gypsum sheathing possesses substantial design strength improvement. The suitability of the current design specifications for the sheathing braced design of CFS structural member is verified. The pull-through failure of the sheathing bracing connection is given due attention. The following conclusions drawn from this study:

1. The gypsum sheathing is adequate for hindering the lateral displacement of the hat shaped CFS wall frame stud. However, the gypsum sheathing failed in pull-through failure in singly and point symmetric CFS wall frame stud indicating the inadequacy of gypsum sheathing board.
2. A detailed analysis is carried out based on the 107 experimental test results to quantify the bracing effect of gypsum sheathing and appropriate suggestions are listed based on the design capacity and failure mode of the CFS wall frame stud.
3. The sheathing bracing design concepts of AISI may include explicit recommendations for provision of sheathing along with various shape and slenderness of the CFS wall frame stud.
4. Further investigation on various design parameters of the CFS wall frame stud is required to develop a demand and supply based effective design method.

Declaration of Competing Interest

None

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