



Using bolted connections for the construction, de-construction and reuse of lightweight exterior infill walls: Experimental study

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Abstract

Bolted connections offer advantages in terms of disassembly and reusability, potentially replacing conventional connections like screws, welds, or chemical bonds. This research investigates the behaviour of bolted connections between lightweight exterior infill walls and beams of primary structural members that are conventionally connected using screws. Although previous studies have investigated bolted connections in different structural members, understanding of the behaviours of these specific connections remains limited. The connections between infill walls and steel beams primarily experience shear loads under serviceability conditions. Therefore, an experimental study was conducted to gain insight into their shear behaviour. The obtained experimental results were analysed using existing predictive equations from design standards that are used across European, North American and Oceanian countries, to identify the most suitable equations for designing such connections.

Keywords Cold-formed steel · Bolted connections · Exterior walls · Reuse · Sustainability · Circular economy

Introduction

Background

The current practice of construction of structures, based on the “take-make-waste” linear economy principle, requires a significant amount of natural resource extraction, waste production and carbon emissions [1]. The necessary activities for construction, such as extracting materials, manufacturing structural members, and transporting them for long distances, affect the Earth’s climate - leading to resource depletion, global warming, and other detrimental impacts [2, 3]. However, the construction of structures is essential in supporting modern life for humans, especially in an era of rapid urban population growth [4]. Thus, we need a different

way of practising construction that minimises the negative impact on our earth while at the same time supporting the construction of essential structures for sustaining modern human life.

In the construction sectors in the UK, several government bodies and industries thus recently incorporated the principles of the circular economy into their practices [5, 6]. These involve actions such as reducing material extraction, minimising waste, reducing interventions (e.g., replacement of exterior walls) by extending lifetimes and reusing and/or recycling products, components and materials [7, 8]. As one such government initiative, the United Kingdom Research and Innovation (UKRI) National Interdisciplinary Circular Economy Research (NICER) program was launched in 2021. The authors of this paper conducted a project at the Interdisciplinary Circular Economy Centre for Mineral-Based Construction Materials (ICEC-MCM), aimed at enabling circular economy principles in the construction of lightweight exterior infill walls for building facades [9]. In this project, circular economy principles involved demounting infill walls from the primary structural members (beams or slabs) after a typical service life of 30 years for reuse in other wall constructions. In the current practice, although many of the components for the exterior lightweight infill walls are durable and could be used for longer than 30 years, those

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components after a typical service life of 30 years are sent to energy-intensive recycling processes or landfills, which are detrimental to our planet in many ways [9]. Reusing the exterior lightweight infill walls can reduce the need for extracting materials, manufacturing, and transportation, thus leading to minimising such detrimental impacts on our planet.

Connection systems for lightweight exterior infill walls

In the UK, lightweight exterior infill walls are widely used in constructing building facades due to their lightness, cost-effectiveness, and ease of construction [9]. Each of the components in the infill wall is connected using screws with different types depending on which components are connected by them [9]. The authors of this article investigated the feasibility of disassembling and reusing several wall components using experimental studies and literature analysis, including connections between cold-formed steel (CFS) members [9], plasterboards and CFS studs [10] and CFS tracks and steel beam (screw connection) [11].

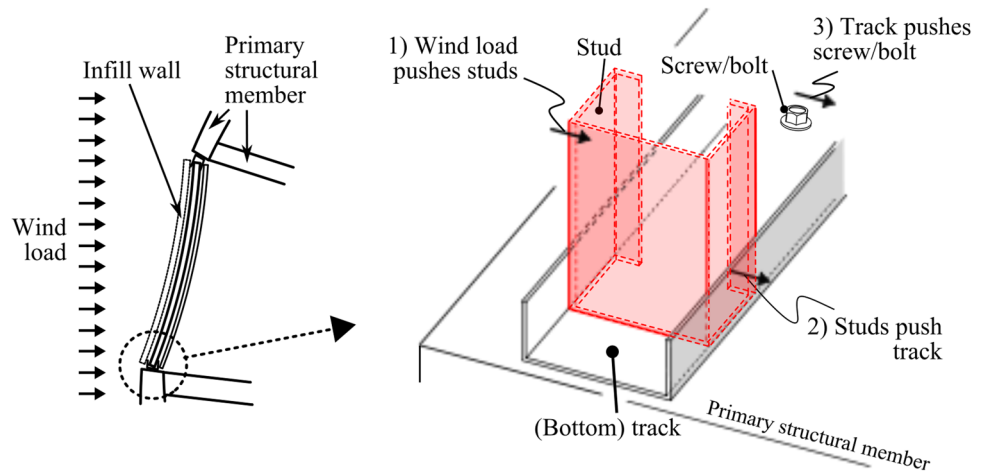
For the construction of exterior lightweight infill walls, current practice uses screws to install the lightweight exterior infill walls to the building frames (see Fig. 1). This paper presents the results from the experimental investigation on the use of bolts for such connections. This is motivated by the previous study by the authors that demonstrated that using bolts can ease the disassembly and reuse of structural members in buildings [12]. Bolts offer advantages over welded, screwed, or chemically bonded connections due to their ease of erection, eliminating the need for welding or high-speed electric screwdrivers [12], as such several studies investigated the use of bolts for connections for CFS members, including beam-to-column connections [13–16], brace-to-gusset plate connections [17], and cord-to-web connections [18], however, there are no recent studies that investigated the use of bolts for connections between the exterior lightweight infill wall and primary structural members.

This paper focuses on the bolted connections between the lightweight exterior infill wall and primary structural members, particularly steel beams. Figure 2 illustrates the behaviour of a considered infill wall, subjected to wind loads, and indicates the location of the connection. As depicted in the

Fig. 1 Lightweight exterior infill walls (Photos taken by authors)



Fig. 2 Connections between lightweight exterior infill walls and primary structural members



figure, the connections (screws or bolts) between the infill walls and primary structural members experience shear loading when subjected to lateral wind forces on the lightweight exterior infill walls. Therefore, this paper focuses on the shear behaviour of these connections. It should be noted that lightweight exterior infill walls comprise various components such as internal and external plasterboards, insulations, cladding, and windows [19]. The connections between these components also impact the construction and deconstruction of the infill walls from the primary structural members. However, the detailed discussions regarding this aspect are beyond the scope of this paper. Interested readers may refer to other publications by the authors [9–11] that addressed such aspects.

Note that the intensity of wind load differs depending on the site conditions where the buildings are constructed and their geometries. Such wind load may be determined based on the relevant Eurocodes (e.g., BS EN 1991-1-4 [20]). Then, the wind load is distributed along the height of the building, and the lightweight exterior infill walls are designed to resist the wind load specific to the height of the building. Furthermore, the design of lightweight exterior infill walls should also account for the weight of certain components, such as cladding.

Literature review on experimental tests

The following literature review focuses on the works that investigated the behaviour of bolted connections with CFS plates. The review is provided to summarise the previous works and establish the innovation of this paper over the existing works.

Previous works used four configurations to carry out shear tests on bolted connections, as illustrated in Fig. 3. Among them, the configuration that is most representative of the behaviour of connections between infill walls and primary structural members is the “Single (lap) shear bolted with different sheet thickness”. The results of the experiment presented later in this paper are based on the experiment using this configuration. The number of investigations into the behaviour of connections using this test configuration is relatively scarce in comparison with other test configurations. Thus, the more popular “Single (lap) shear bolted with same sheet thickness” was also utilised to consider how different test configurations may lead to different results. The following analysis of literature focuses on the works that utilised these two test configurations.

Zadanfarrokh and Bryan [21] conducted experiments to investigate the shear behaviours of CFS plates with sheet thicknesses ranging between 1.5 and 3.2 mm. The test specimens were designed with specific geometries, including plate width and distance from the bolt hole to the plate edge, to ensure bearing failure of the connections. The tests involved bolting a single CFS plate to a 9 mm thick hot-rolled steel plate. The results demonstrated that the strength of the bolted joint matched that of the thinnest connected sheet, i.e., the CFS plates. Interestingly, the thickness of the hot-rolled steel plate did not affect the strength of the tests, as the specimens failed at the same loads as when two sheets of equal thickness were used. Additionally, the study observed that using a washer on one CFS plate increased the bearing strength by 1.14 times compared to cases without a washer.

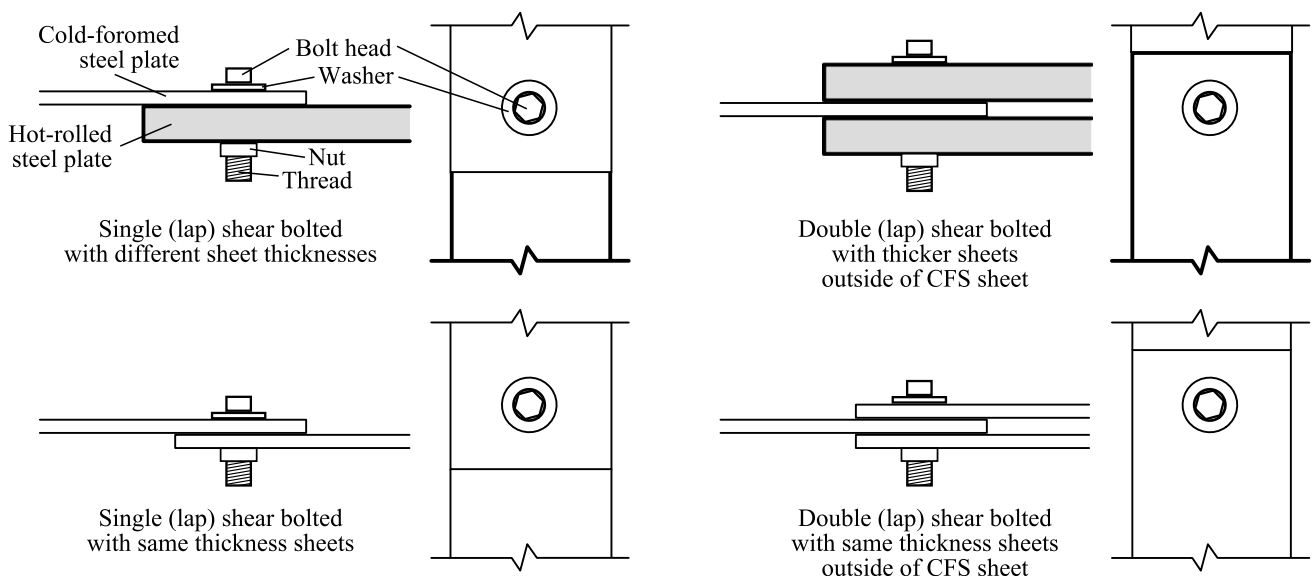


Fig. 3 Typical test configurations of bolted connections for CFS plates

Wallace and LaBoube [22] conducted experiments to investigate the shear behaviours of CFS plates with sheet thicknesses of 0.64 and 1.38 mm. The tests were carried out on a single CFS bolted to another CFS plate that had the same or different thicknesses. The study found that the use of washers in bolted connections had a significant impact on the strength of the connections, particularly when bearing was the mode of failure. The geometries of the test specimens, such as plate width and distance from the bolt hole to the plate edge, were specifically determined to induce bearing failure. While there were no specific discussions on the results comparing thin-thin plates and thin-thick plates, the overall comparisons showed that the peak strength of thin-thick plates was slightly higher than that of thin-thin plates, regardless of the use or absence of washers. Note that the study conducted by Wallace and LaBoube [22] is less directly relevant to the current investigation since it did not utilize hot-rolled steel for thick plates, unlike the study by Zadanfarrokh and Bryan [21] which used 9 mm hot-rolled steel for thick plates.

Several studies have examined the behaviour of steel plate connections using the “Single (lap) shear bolted with same sheet thickness” configuration [23, 24]. Other studies have compared the results from different test configurations, such as “Single (lap) shear bolted with same thickness sheets” and “Double (lap) shear bolted with thicker sheets outside of CFS sheet,” to investigate the behaviour of CFS plates. However, these studies were not reviewed in detail in this article as they do not provide insights into the behaviour of bolted connections using thin-thick plates, specifically the combination of CFS plate on one side and hot-rolled steel plate on the other side, which is the focus of the current study.

Innovation in this article

In the body of published work on the behaviour of bolted CFS plates, there have been a scarce number of experimental works that can be used in the design of bolted connections between lightweight exterior infill walls and the primary structural members. While the behaviour of bolted connections for such application can be experimentally tested most representatively by using the test configuration “Single (lap) shear bolted with different sheet thickness” (see Fig. 3) using hot-rolled steel plate on one side, there is only one experimental works that reported such results [21]. Although the results reported in the referenced article may be helpful in understanding the connection behaviour and developing the design guidance, detailed information is missing, such as the exact number of tested samples, load-deformation data of each test, and the exact cases of tested steel plate samples. Also, Zadanfarrokh and Bryan [21] used Grade 4.6 bolts whose mechanical properties are not commonly

used nowadays while this study used Grade 8.8 bolts since this is the most common grade currently utilised in practice in the U.K. Additionally, a recent study [24] reported that the strength of connections varies depending on how bolts behave under shear load. For example, in their study, the strength was recorded lower when the bolt rotation (due to steel plate curling) was allowed at large connection deformation (when using the configuration, “Single (lap) shear bolted with same sheet thickness”) than when the bolt rotation was fixed (when using the configuration, “double (lap) shear bolted with thicker sheets outside of CFS sheet”). This is however overlooked in the study of Zadanfarrokh and Bryan [21] where the curling of CFS members was constrained by forming lips around the connection area. Additionally, in the U.K., the common thickness for CFS steel plates for exterior infill walls nowadays are 1.2, 1.6 and 2.0 mm, however, none of the published studies used these thicknesses when they investigated the bolted connection behaviour. This research, therefore, investigates the behaviour of such bolted connections used as connectors in thin and thick plates, in which the thin steel plate (i.e., CFS plate) represents the track of lightweight exterior infill walls and the thick steel plate (i.e., hot-rolled steel plate) represents the primary structural members (beams). The paper also analyses the suitability of predictive equations available in the design standards to investigate which equations best suit for the design of bolted connections between lightweight exterior infill walls and the primary structural members.

Experimental program

Tests of coupons

Tensile coupon tests were conducted to determine the material properties of the CFS tracks of grade S390 galvanised with Z275 zinc coating and hot-rolled steel plates of grade S355 (uncoated). Three to four coupons were extracted from cold-formed steel tracks perpendicular to the length of the web with 1.2, 1.6 and 2.0 mm nominal thicknesses. The nominal dimensions of the tensile coupons are illustrated in Fig. 4. The coupons were tested based on BS EN ISO 6892-1 [25] where an axial deformation rate of 1 mm/minute was applied for all coupons. Note that BS EN ISO 6892-1 [25] allows the use of constant strain rate if the primary purpose of the tensile coupon test is only to determine the tensile strength (i.e., ultimate stress, f_u). A 100 kN Instron 3382 testing machine was used to perform the tests for CFS coupons, and a 600 kN Instron 5989 testing machine was used to perform the tests for hot-rolled steel coupons, along with the strain gauges (with a gauge length of 50 mm) which was mounted on the coupon, and the data were collected every 0.02 s. The test results are listed in Table 1. The specimens

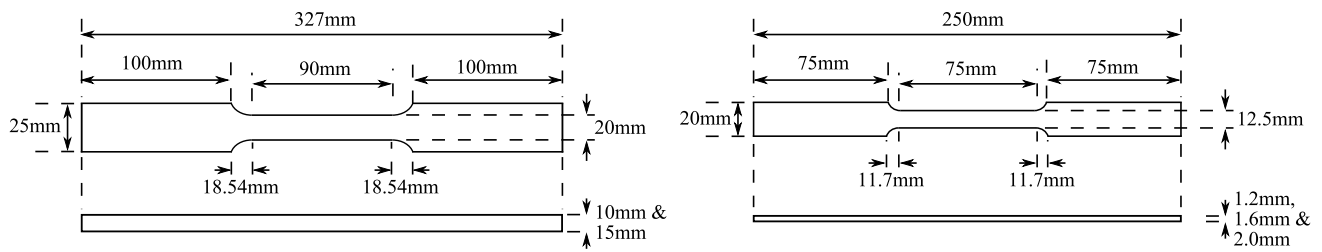


Fig. 4 Nominal dimensions of steel tensile coupons (left: hot-rolled steel; right: CFS)

Table 1 Results of coupon tests and shear connection tests

Specimen	w (mm)	Plate 1			Plate 2			P _u (kN)
		t ₁ (mm)	f _{y1} (MPa)	f _{u1} (MPa)	t ₂ (mm)	f _{y2} (MPa)	f _{u2} (MPa)	
1.2-10-w-50	50	1.169	348	437	10	403	531	15.7
1.2-10-wo-50	50	1.169	348	437	10	403	531	14.5
1.2-15-w-50	50	1.169	348	437	15	392	536	15.2
1.2-15-wo-50	50	1.169	348	437	15	392	536	13.6
1.2-1.2-w-50	50	1.169	348	437	1.169	348	437	13.2
1.2-1.2-wo-50	50	1.169	348	437	1.169	348	437	11.9
1.6-10-w-50	50	1.567	439	528	10	403	531	28.5
1.6-10-wo-50	50	1.567	439	528	10	403	531	22.0
1.6-15-w-50*	50	1.567	439	528	15	392	536	25.0
1.6-1.6-wo-50*	50	1.567	439	528	1.567	439	528	19.9
1.6-10-w-75	75	1.554	567	624	10	403	531	28.5
1.6-10-wo-75	75	1.554	567	624	10	403	531	24.0
1.6-15-w-75	75	1.554	567	624	15	392	536	32.6
1.6-15-wo-75	75	1.554	567	624	15	392	536	23.8
1.6-1.6-w-75	75	1.554	567	624	1.554	567	624	23.8
1.6-1.6-wo-75	75	1.554	567	624	1.554	567	624	23.5
2.0-10-w-50	50	1.744	330	382	10	403	531	24.9
2.0-10-wo-50	50	1.744	330	382	10	403	531	19.1
2.0-15-w-50	50	1.744	330	382	15	392	536	24.3
2.0-15-wo-50	50	1.744	330	382	15	392	536	19.0
2.0-2.0-w-50	50	1.744	330	382	1.744	330	382	18.4
2.0-2.0-wo-50	50	1.744	330	382	1.744	330	382	16.6
2.0-10-w-75	75	1.750	321	388	10	403	531	28.9
2.0-10-wo-75	75	1.750	321	388	10	403	531	22.8
2.0-15-w-75	75	1.750	321	388	15	392	536	29.0
2.0-15-wo-75	75	1.750	321	388	15	392	536	22.0
2.0-2.0-w-75	75	1.750	321	388	1.750	321	388	21.7
2.0-2.0-wo-75	75	1.750	321	388	1.750	321	388	21.1

The specimens with * give average of two ultimate strength values from shear connection tests (others give average of three ultimate strength values)

in Table 1 were labelled such that the plate nominal thicknesses for plate 1 and plate 2, the existence of washer (w for “with washer” or wo for “without washer”) and plate width could be identified. For example, the specimen at the top of Table 1, “1.2” and “10” stand for the nominal thicknesses of plates 1 and 2, respectively, in millimeters, “w” stands that the specimen had a washer, and “50” stands for

the 50 mm width of the specimen. Note that the difference in tensile strength observed among CFS coupons of the same thickness may be attributed to variations between tracks with different product codes provided by the steel track supplier. The yield stress, f_y , was also determined and listed in Table 1. The yield stress was determined by (i) drawing the stress-strain curve from the experiment, (ii) drawing a line

parallel to the linear portion of the curve and at a distance from it equivalent to the strain of 0.2%, and (iii) determining the stress that corresponds to the point at which the lines intersect.

Tests of bolted connections

Overview

A test program on bolted connections between the steel plates has been carried out in the George Earle testing laboratory at the University of Leeds. Figure 5 shows the machine set up for the tests. In total, 82 bolted connections were tested.

In these tests, three parameters were varied: CFS plate thickness (1.2, 1.6 and 2.0 mm), CFS plate width (50 and 75 mm), and the existence of (i.e., with/without) washer. The bolt grade is 8.8 with a 16 mm diameter. According to the BS EN 1993-1-8 [26], the nominal yield strength of this bolt is 640 MPa and the nominal ultimate tensile strength is 800 MPa. The bolts were placed in holes with a diameter 1 mm larger than the bolt diameter. Figure 6 presents the geometries of tested specimens. The bolts were installed using two wrenches and tightened moderately. Since the failure mode of interest is connection bearing, considering it as the most likely failure mode for the geometries shown in Fig. 2, the connection configurations were determined to eliminate other failure modes based on Table 8.4 in BS EN 1993-1-3 [27]. All specimens were fabricated by cutting CFS tracks. All bolted specimens were tested in a 600 kN Instron 5989 testing machine (Fig. 5) with a 660 kN load cell. The specimens were gripped at each end using mechanical grips. For single-lap connections with a hot-rolled steel plate at one side, to reduce the eccentricity induced in the

testing rig, a 95 mm long steel packing plate was used for testing specimens with the same thickness (either 10 or 15 mm) and width (either 50 or 75 mm) as those paired hot-rolled steel plates. All tests were performed using the displacement-controlled upper grip of the testing machine with a constant stroke rate of 1 mm/min.

Results

The results of the experiments, in terms of the ultimate strength (P_u) of the connections, are presented in Table 1. The values in the table are the averages of three identical tests, except for some cases with asterisk which are the averages of two tests.

In general, the test results demonstrated that: (i) the use of a washer increased the peak strength of connections, (ii) the thickness of the hot-rolled steel plates (10 or 15 mm) did not affect the strength of the connection, (iii) the strength of the connection dropped when the CFS plates were used in both sides of the connections. The observation (i) was previously reported by Zadanfarrokh and Bryan [21] and Wallace and LaBoube [22] for other mechanical and geometrical specimens. The observation (ii) is due to the much larger thickness of hot-rolled steel plates of both thicknesses (10 and 15 mm) than the connected CFS plates, which led to the concentrations of deformation in the CFS plates. The observation (iii) is attributed to the tilting of bolts, as supported by the previous study conducted by Ding et al. [24] on other mechanical and geometrical specimens. Their research considered the behaviour of connections of two CFS plates using bolts and demonstrated that the presence of tilting bolts led to a reduction in the peak shear strength of the bolted connections.

Fig. 5 Test setup for shear connection test



Universal Testing Machine for shear connection test



Specimen fixed by the mechanical grips



Specimen during the test

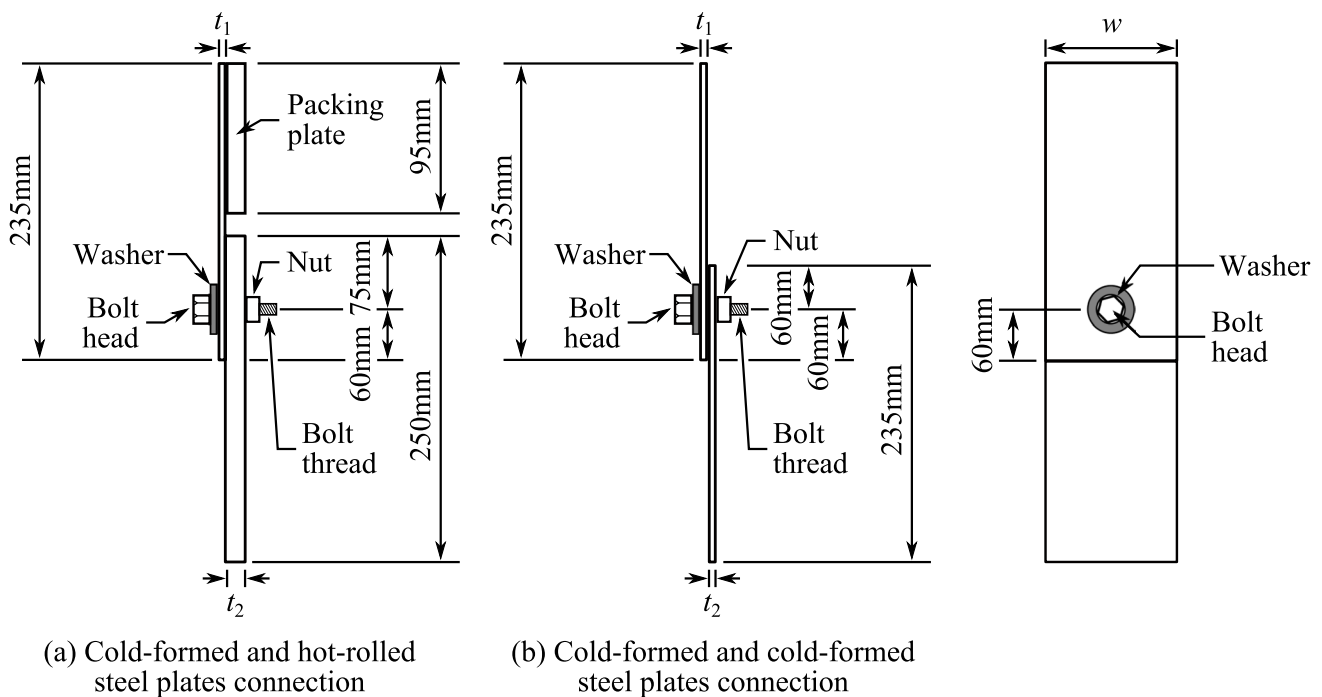


Fig. 6 Geometries of tested specimens

Evaluation of existing equations (Eurocode, AISI, AS/NZS)

This section investigates whether the existing equations for the prediction of bolted connections in the design standards can accurately predict the strength and failure mode of the tested bolted connections. The section first provides the overview of the existing predictive equations for different modes of failure from BS EN 1993-1-3 [27], AISI S100-16 [28] and AS/NZS 4600:2018 [29]. Each standard provides equations to predict peak connection strength for different failure modes, which are illustrated in Fig. 7. The section then follows discussions on which standard is the most suitable for predicting the failure mode and peak strength for the considered bolted connections.

Prediction of connection failure using BS EN 1993-1-3

In Table 8.4 in BS EN 1993-1-3 [27], the prediction of the strength of bolted connections is provided for bearing resistance, net-section resistance, and shear resistance. The shear resistance in this code is used when the bolt is the critical part in the connection to resist the force (i.e., considering bolt shear failure). In the current study, the bolt failure did not occur as the bolt is in Grade 8.8 with high strength as explained previously in this paper, thus this is not considered in this sub-section. The following equations are used to evaluate the strength of connection when bearing resistance is considered:

$$P_{EC,B} = 2.5 \cdot \alpha_b \cdot k_t \cdot f_u \cdot d \cdot t \tag{1}$$

$$k_t = \begin{cases} \frac{(0.8t+1.5)}{2.5} & 0.75 \text{ mm} \leq t \leq 1.25 \text{ mm} \\ 1.0 & t > 1.25 \text{ mm} \end{cases} \tag{2}$$

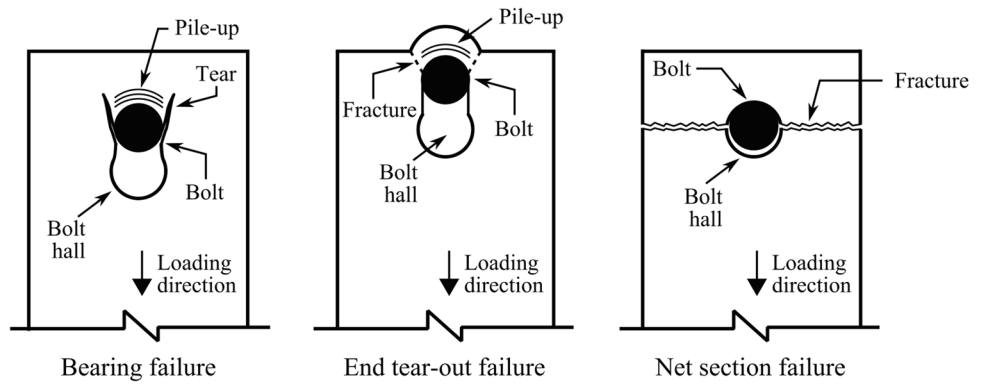
$$\alpha_b = \min\left(1.0, \frac{e_1}{3d}\right) \tag{3}$$

where f_u is the ultimate tensile stress of material (see Table 1), d is the nominal diameter of the bolt ($= 16 \text{ mm}$ for all cases), e_1 is the end distance from the centre of the bolt hole to the end of the plate in the direction of load transfer, and t is the thickness of the thinner connected part or sheet (t_1 in Table 1). The following equations are used to evaluate strength of the connection when net-section resistance and single bolt at the connection are considered:

$$P_{EC,N} = \left\{ 1 + 3 \left(\frac{d_h}{2e_2} - 0.3 \right) \right\} \cdot A_{net} \cdot f_u \tag{4}$$

where d_h is the nominal diameter of the bolt hole, e_2 is the half of the width of the plate ($= w/2$ in Table 1), and A_{net} is the net-cross sectional area of the connection ($= t \cdot (w - d_h)$). In this study, the Eqs. (1) and (4) are used to calculate connection strength, and the one that gave a smaller value was considered as the strength of the connection and the failure mode, i.e., $P_{EC} = \min(P_{EC,B}, P_{EC,N})$. The predicted failure mode and the connection strength (used to normalize

Fig. 7 Failure modes as defined in design codes



connection strength from the experiment) are reported in Table 2. Note that the use of the above equations for the prediction of the strength of the specimens with a width of 50 mm violates the rules in Table 8.4 in BS EN 1993-1-3

[27] only marginally. Also, note that the above prediction equations in BS EN 1993-1-3 [27] do not distinguish between a double (lap) shear and a single (lap) shear connection nor do not consider the modification of strength when

Table 2 Evaluation of existing equations and failure modes

Specimen	Failure mode from experiment	P_u/P_{EC}	Failure mode from EC	P_u/P_{AISI}	Failure mode from AISI	$P_u/P_{AS/NZS}$	Failure mode from AS/NZS
1.2-10-w-50	Net Sect. (3)	0.79	Bearing	1.00	Net-section	1.10	Net-section
1.2-10-wo-50	Net Sect. (3)	0.73	Bearing	0.92	Net-section	1.01	Net-section
1.2-15-w-50	Net Sect. (3)	0.76	Bearing	0.97	Net-section	1.06	Net-section
1.2-15-wo-50	Net Sect. (3)	0.68	Bearing	0.87	Net-section	0.95	Net-section
1.2-1.2-w-50	Net Sect. (3)	0.66	Bearing	0.84	Net-section	0.92	Net-section
1.2-1.2-wo-50	Bearing (2), Net Sect. (1)	0.60	Bearing	0.76	Net-section	0.83	Net-section
1.6-10-w-50	Bearing (1), Net Sect. (2)	0.86	Bearing	1.12	Net-section	1.23	Net-section
1.6-10-wo-50	Bearing (2), Net Sect. (1)	0.66	Bearing	0.86	Net-section	0.95	Net-section
1.6-15-w-50*	Net Sect. (2)	0.76	Bearing	0.98	Net-section	1.08	Net-section
1.6-1.6-wo-50*	Bearing (1), Net Sect. (1)	0.60	Bearing	0.78	Net-section	0.86	Net-section
1.6-10-w-75	Net Sect. (3)	0.73	Bearing	0.62	Bearing	0.62	Bearing
1.6-10-wo-75	Bearing (2), Net Sect. (1)	0.62	Bearing	0.69	Bearing	0.69	Bearing
1.6-15-w-75	Bearing (3)	0.84	Bearing	0.71	Bearing	0.71	Bearing
1.6-15-wo-75	Bearing (3)	0.61	Bearing	0.69	Bearing	0.69	Bearing
1.6-1.6-w-75	Bearing (3)	0.61	Bearing	0.52	Bearing	0.52	Bearing
1.6-1.6-wo-75	Bearing (3)	0.61	Bearing	0.68	Bearing	0.68	Bearing
2.0-10-w-50	Net Sect. (3)	0.93	Bearing	1.22	Net-section	1.33	Net-section
2.0-10-wo-50	Net Sect. (3)	0.72	Bearing	0.93	Net-section	1.02	Net-section
2.0-15-w-50	Net Sect. (3)	0.91	Bearing	1.19	Net-section	1.30	Net-section
2.0-15-wo-50	Bearing (1), Net Sect. (2)	0.71	Bearing	0.93	Net-section	1.02	Net-section
2.0-2.0-w-50	Net Sect. (3)	0.69	Bearing	0.90	Net-section	0.98	Net-section
2.0-2.0-wo-50	Bearing (2), Net Sect. (1)	0.62	Bearing	0.81	Net-section	0.89	Net-section
2.0-10-w-75	Bearing (3)	1.06	Bearing	0.89	Bearing	0.89	Bearing
2.0-10-wo-75	Bearing (3)	0.84	Bearing	0.93	Bearing	0.93	Bearing
2.0-15-w-75	Bearing (3)	1.07	Bearing	0.89	Bearing	0.89	Bearing
2.0-15-wo-75	Bearing (3)	0.81	Bearing	0.90	Bearing	0.90	Bearing
2.0-2.0-w-75	Bearing (3)	0.80	Bearing	0.67	Bearing	0.67	Bearing
2.0-2.0-wo-75	Bearing (3)	0.78	Bearing	0.86	Bearing	0.86	Bearing

(The specimens with * give an average of two ultimate strength values; the values in the parenthesis are the number of observations)

using washers. Also, while BS EN 1993-1-3 [27] does not distinguish the “Bearing failure” and “end tear-out” failure modes, previous work by Teh and Uz [23] explained that they are different failure modes and must be treated differently as they affect the strength of the bolted connections differently.

Prediction of connection failure using AISI S100-16

In Section J3.3.1 in AISI S100-16 [28], the prediction of the bearing strength of bolted connections is provided as follows:

$$P_{\text{AISI,B}} = C \cdot m_f \cdot d \cdot t \cdot f_u \tag{5}$$

where C is the bearing factor, which is determined in accordance with Table J3.3.1-1 in AISI S100-16 [28], and m_f is the modification factor for bearing connection, which is determined according to Table J3.3.1-2. Factor C accounts for the degrees of bearing occurring at the connection depending on the bolt diameter and steel plate thickness. Factor m_f accounts for the existence of washers and how the CFS plates are confined (i.e., single lap or double lap shear connections). In this study, $C=3.0$ is used when the nominal thickness of the CFS plate is 2.0 mm, and $C=4-0.1(d/t)$ is used when other thicknesses are considered (nominal thicknesses of 1.2 and 1.6 mm). Additionally, $m_f=1.0$ was used when the washers were used and $m_f=0.75$ was used when the washers were not used.

In Section J6.1 in AISI S100-16 [28], the prediction of shear rupture strength of bolted connections is provided as follows:

$$P_{\text{AISI,N}} = 1.2 \cdot t \cdot e_{\text{net}} \cdot f_u \tag{6}$$

where e_{net} is the clear distance between the end of material and the edge of the bolt hole ($e_{\text{net}}=51.5$ mm for all CFS plates; see Fig. 6). Also, in Section J6.2 in AISI S100-16 [28], the prediction of tension rupture strength of bolted connections with a single bolt hole is provided as follows:

$$P_{\text{AISI,SO}} = (A_g - d_h \cdot t) \cdot \left(0.9 + 0.1 \cdot \frac{d}{w}\right) \cdot f_u \tag{7}$$

where A_g is a gross area ($=t \cdot w$). In this study, the Eqs. (5), (6) and (7) are used to calculate connection strength, and the one that gave the smallest value was considered as the strength of the connection and the failure mode ($=P_{\text{AISI}}$). The predicted failure mode and the connection strength (used to normalize connection strength from the experiment) are reported in Table 2. Note that the “AISI” stands for AISI S100-16 [28].

Prediction of connection failure using AS/NZS 4600

In Clause 5.3.4.2 of AS/NZS 4600 [29], the bearing strength provision from AISI S100-16 [28] (see Eq. (5)) is adopted by AS/NZS 4600:2018 [29] (also see [30, 31]). For shear-out strength, the following equation is used [29]:

$$P_{\text{AS/NZS,SO}} = t \cdot e_1 \cdot f_u \tag{8}$$

For net section tension strength, the strength is specified in Clause 3.2.2 and Clause 5.3.3 of the AS/NZS [29] (also see [32]), where the strength is determined by the minimum of the three calculated values:

$$P_{\text{AS/NZS,N}} = \min\left(A_g \cdot f_y, 0.85 \cdot k_1 \cdot A_{\text{net}} \cdot f_u, (A_g - d_h \cdot t) \cdot \left(0.9 + 0.1 \cdot \frac{d}{w}\right) \cdot f_u\right) \tag{9}$$

where k_1 is a correction factor, which in this study is taken as 1 ($k_1=1.0$ per [32]). Note that the third equation from the left in Eq. (9) is identical to the Eq. (7) of AISI S100-16 [28]. In this study, the Eqs. (5), (8) and (9) are used to calculate connection strength, and the one that gave the smallest value was considered as the strength of the connection and the failure mode ($=P_{\text{AS/NZS}}$).

Comparisons between test results and predictions

When the CFS plates are restrained from curling, the failure mode might follow one of the failure modes defined in the design standard as shown in Fig. 7 [33]. Note that as the experiment did not restrain the samples from curling, the failure modes observed involved curling of the edge of the CFS plates for all tested specimens except just three cases (one in 2.0-10-w-50 and two in 2.0-15-w-50). When both plates were CFS plates, the tilting of bolts was also observed. These might have affected the accuracy in the prediction of strength and failure mode. The experiment conducted in this study did not show the “end tear-out” failure mode and the equations in the design standard did not give the minimum strength from this failure mode, as shown in Table 2. The reason the “end tear-out” failure mode did not occur was that the distance between the bolt hole and the end of the plate was sufficient to prevent this failure mode from occurring. There were also some test cases where the identification of failure mode was difficult as the test samples showed crack propagations horizontally with the large bearing of bolt head into the cold-formed steel plate. Photos of the specimens that experienced bearing and net section failures are shown in Fig. 8.

The comparisons between test results and the predictions from the design standard show that, on average, the use of AS/NZS 4600 [29] gave the most accurate peak strength predictions among all the design standards (average of P_u/P_{EC} ,



Specimen 1.2-15-w-50
Net section failure



Specimen 2.0-10-wo-75
Bearing failure

Fig. 8 Net section and bearing failure modes

P_u/P_{AISI} , $P_u/P_{AS/NZS}$ are 0.75, 0.86 and 0.91, respectively). This is because AS/NZS 4600 [29] uses the most accurate equations to predict bearing strength (along with the AISI standard) and net-section failure strength. The EC code could not distinguish the different failure modes of bearing and net-section failures accurately, leading to the prediction of bearing failure mode for all cases. The equations in AISI [28] and AS/NZS 4600 [29] for bearing strength predictions consider the existence of washers while the EC standard does not. The considerations of washers in predicting the connection strength led to a moderate improvement in predicting the connection strength.

Limitations of this work

The structural load at the connection between lightweight exterior infill walls and the beams are primarily loaded in shear as explained in Fig. 2. The experiment was conducted to load the specimens until the failure was observed and their strength dropped to near zero. However, when walls are subjected to the wind load in out-of-plane directions, the connections may be loaded in a more complex manner – not just in shear. Such complex behaviour may affect the strength of the connections between lightweight exterior infill walls and primary structural members. To observe such complex behaviour, an additional experiment using full-scale wall specimens or the entire building models needed to be conducted. This is a limitation of this study that may be worthy of future investigation.

Conclusions

This paper investigated the use of bolts for connecting lightweight exterior infill walls to beams of primary structural frame members for the purpose of easing the construction, deconstruction and reuse of lightweight exterior infill walls. The infill walls are subjected to wind load in their out-of-plane direction. The load is transferred to the primary structural members of beams or slabs. This load transfer is enabled when the connections between the infill walls and beams or slabs are designed to resist the expected shear force. Thus, the study conducted experiments to assess the shear strength of these connections, using CFS plates to represent the infill walls and hot-rolled steel plates to represent the steel beams. The experiments yielded the following observations: (i) the use of washers increased the strength of the connections, (ii) the different thicknesses of the hot-rolled steel plates (10 or 15 mm) did not affect the strength of the connections, and (iii) the use of CFS plates on both sides of the connections reduced the strength. The experimental results were compared with the connection strength predicted by equations from three different design standards. Examination of predictive equations for bolted shear connections in different design standards revealed that the equations in AS/NZS 4600 [29] are recommended for designing bolted connections in infill wall construction as it gave the most accurate peak strength predictions among all the design standards. The present study was limited to the consideration of connections. Further research, especially the large-scale experiment, is warranted to establish the use of bolted connections between the lightweight exterior infill walls and primary structural members.

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Data availability The data that support the findings of this study are available from the corresponding author upon reasonable request.

Declarations

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

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