Innovative Australasian cold-formed steel platform framing collector beams for multi-storey residential buildings

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ABSTRACT
The need for population density in major New Zealand cities is driving demand for mid-rise residential buildings. Cold-formed steel framing offers a potentially very good solution for these buildings. However, achieving this requires understanding four-to-six-storey system behaviour under severe seismic actions. In a multi-storey CFS building, the stacked, strap-braced shear walls utilising energy dissipation through capacity-based design principles form an integral part of the internal and external wall panel system. Under the earthquake action with gravity loading, large compression loads are transferred through the columns (chords) of the top wall into the flooring system and through to the bottom walls. A detail comprising a joist stiffener has been developed to increase the compression strength of a platform framing connection in an Australasian CFS construction, considering internal and external wall systems. Recommendations to prevent bearing failure through experimental tests are presented in the paper. To avoid the large compression loads from the top storeys crushing the floor sheathing, which runs under the walls, bonded materials have been used in the northern hemisphere. However, steel sheet bonding with flooring materials is not a common construction in the Australasian light gauge steel industry. Therefore, an innovative configuration of collector beams each for the internal and external wall systems for a strong diaphragm action has been developed.

1 INTRODUCTION

Significant advantages of Cold-Formed Steel (CFS), such as a high strength-to-weight ratio, excellent dimensional stability and the capability to resist high lateral loads, led to its increased usage in low-to-mid-rise residential buildings in high seismic areas. In New Zealand and Australia, the use of high strength (> 500 MPa) and thin gauge (< 0.9 mm) is significantly different to Northern Hemisphere steel, which is higher gauge (thicker) and lower in strength. This combination results in the thin gauge CFS members being more susceptible to buckling under compression and bearing.

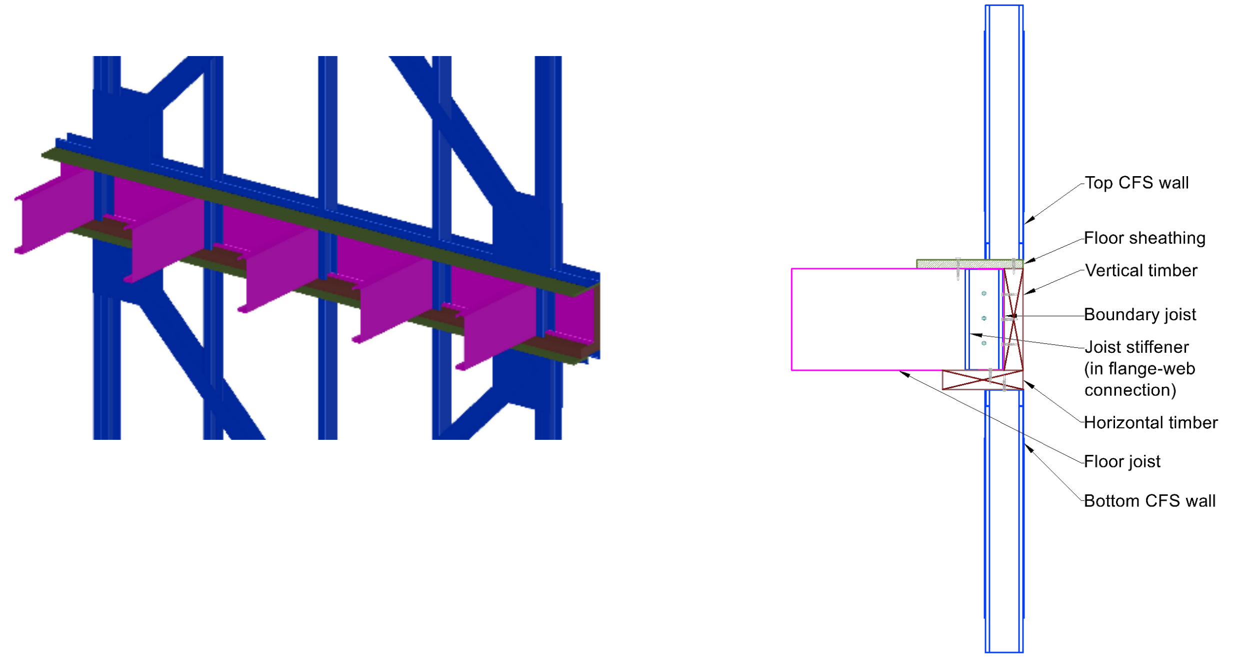
Typically, two methods are used for wall/floor/wall construction of mid-rise light framing: platform framing and ledger framing. In platform framing, the floor joist assembly is stacked on top of the track of the bottom wall, and the floor sheathing runs underneath the bottom tracks of the top wall. This method interrupts the continuity of the wall panel system, allows for sequential storey construction and is also standard practice for two and three storey construction, hence well-known and accepted by the industry. Figure 1 shows the platform framing configuration for an exterior and interior wall system as proposed and is being tested in this project at the University of Auckland.

Ledger framing construction is widely used in the Northern Hemisphere. In this framing, the walls are stacked directly on top of each other, with the floors supported by ledges connected to the sides of the walls. In balloon framing, which is a type of ledger framing, the joist assembly is connected to angle support fixed to the continuous two-storey studs. Despite the advantages of a direct load path from an upper wall to a lower wall in ledger and balloon framing, the fire-induced movement at the floor-to-wall intersection and the fire-resisting linings pose a risk of failure in severe fire, especially in the thin gauge material used locally. These framings also necessitate the use of bonded floor panels to achieve the required diaphragm action. Results from an extensive seismic and fire study on a ledger framed six-storey CFS building with gypsum boards and cement boards bonded with thin steel sheets led by Hutchinson et al. (2021); Wang and Hutchinson (2021) showed detachment of ceiling gypsum boards, shrinkage and cracking of floor cement boards, and excessive floor deflection due to diaphragm sagging. The sheet steel bonded layer of cement boards likely prevented floor deflections. However, such steel sheet bonded flooring and sheathing material is not available in New Zealand.

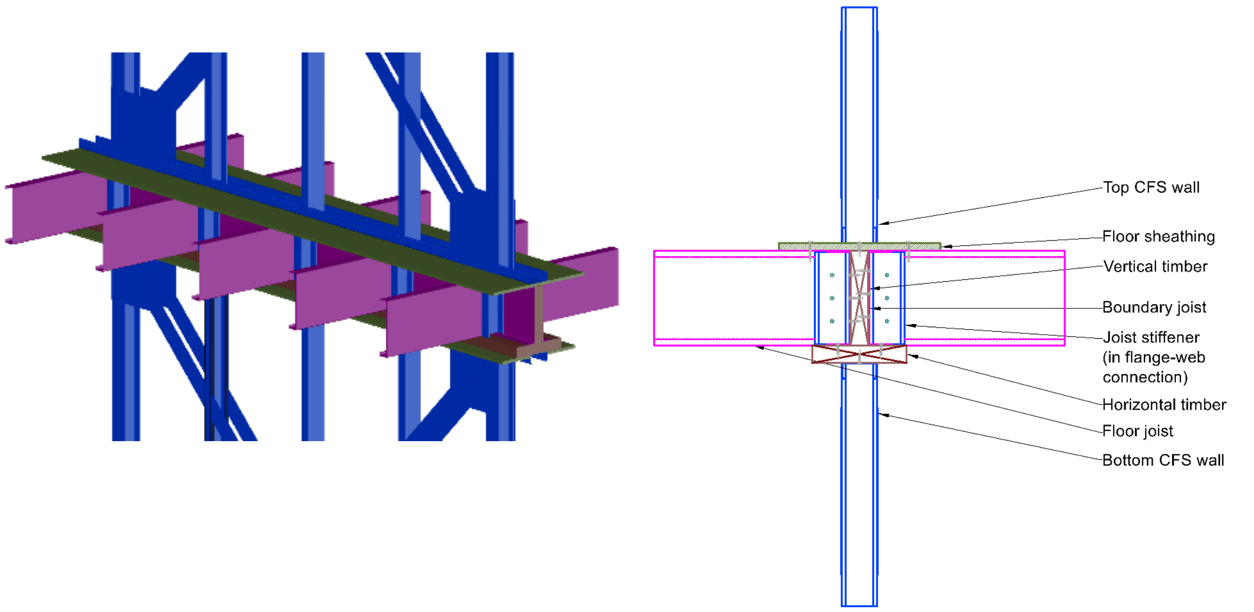
For a seismic-force resisting system in a multi-storey building, the CFS strap braced shear walls are designed on a capacity-based principle, with the tension strap braces being the primary element. The chords of these CFS shear walls transfer large compression forces to the bottom-storey load-bearing walls due to combined earthquake-induced and gravity system loads. The accumulated compression forces are significantly larger than the tension forces resulting from the lateral forces. To understand the behaviour of the platform framing load paths and the interaction of wall/floor/wall subassembly, ten compression tests have been undertaken at the University of Auckland. A full-scale test programme is being developed to observe the tension uplift and wall/floor/wall performance in lateral loading and vertical loading at the Structures Testing Laboratory of the University of Auckland.

Previous studies on CFS platform framing construction have been conducted by Fox (2003); Fox and Brodland (2004); Fox (2005); Fox, Knill, and Schuster (2012) to observe the interaction of channel sections and clip angles as bearing stiffeners to the floor joist system. Most of these experimental and numerical studies provided the basis for bare wall frames and their floor assembly, which are mostly the non-load-bearing walls in a multi-storey platform framing. However, little is known about the performance of joist stiffeners (also known as bearing stiffeners) under compression chords of a CFS shear wall with a floor joist system. These compression load-carrying chords, separated by the floor system, necessitate a stiffened direct load path from the upper wall to the lower wall in each storey of a multi-storey building. A detailed study by Fox (2002) has shown that the floor joist system's stiffness depends on the web-crippling capacity of the joists and stiffeners and the axial capacity of the stiffeners when the bearing stiffeners are connected between the flanges of the floor joists.

The joist stiffeners in the current study are in a web connection with the floor joists and are in between the flanges of the boundary joists. When the joist stiffeners are web-to-web connected to the floor joists, only half of the joist stiffener is in bearing contact with the inside flanges of the boundary joists. The results discussed in the current study showed that the back-to-back chords of a shear wall under a high compression load crushed the flanges of the boundary joist, resulting in the joist stiffener slipping and failing in bearing. A web-to-flange connection of the joist stiffener with the boundary joist with a five-millimetre bearing plate increased the system stiffness and prevented the bearing failure of the stiffener, as described and illustrated in section 3.



(a) Exterior wall SIDE VIEW



(b) Interior wall SIDE VIEW

Figure 1: Proposed platform framing for CFS shear walls

2 MATERIALS AND EXPERIMENTAL METHODS

2.1 Specimen configuration

A configuration of external and internal shear walls was developed for the wall/floor/wall connection. Timber packers were introduced between the boundary joists (as vertical timber) and on top of the top track of the bottom wall (as horizontal timber). Four combinations with respect to timber packers were proposed for the wall system: (a) with vertical and horizontal timber, (b) with only horizontal timber, (c) no timber (a northern hemisphere floor assembly) and (d) with only vertical timber. Studies by Natário, Silvestre, and Camotim (2014); Natário, Silvestre, and Camotim (2017) show that under high stress concentrations, flanges of CFS beams tend to crush, which is often mistaken as web-crippling. Therefore, the horizontal timber beam was expected to provide bearing support for the U-section boundary joist under the compression load-carrying back-to-back chords. In addition to thermal insulation, vertical timber was also introduced to suppress web crippling under compression. Oriented strand board (OSB) was adopted as the flooring material, which was also considered a part of the diaphragm action in the wall/floor/wall system for a platform system. Including the timber packers, the floor joist system consisted of the boundary joist running parallel to the shear wall and the floor joist (abbr. FJ) perpendicular to the shear wall and in alignment with the chords. The joist stiffeners (abbr. JS) had a web-to-web connection with the floor joists under each chord and stud.

Ten experimental tests were performed on a mock-up of a CFS strap-braced wall connected to the proposed floor assembly. The mock-up of the CFS strap-braced walls was adopted from the results of the experimental tests conducted at the University of Auckland by Lotfi Rad et al. (2023). These CFS walls resulted in enhanced performance as a lateral force-resisting system for mid-rise residential buildings under cyclic and constant vertical loading. Excellent seismic performance was observed by the both-sided strap-braced walls, which was also validated in a numerical study by Karmakar, Clifton, and Lim (2023).

2.2 Material properties and experimental set-up

The material properties of all the members (chords, tracks and studs) of the mock-up walls conformed to AS/NZS 4600 (2018). Since the both-sided strap braces were designed to yield, they were of grade G250 and 100 mm wide. The details of the material properties used for the compression tests are summarised in Table 1. Speedfloor (2018) joist system was used for the lipped C-section floor joist and U-section boundary joist.

The 20 mm thick OSB panels were connected to the floor joist system using 10g × 50mm countersunk truss head screws. Type 17 screws were used for all other steel-to-timber and floor joist system connections. All the screws of the floor joist system were spaced at 80 mm centres. The joist stiffener had a three-point connection to the boundary joist with type 17 screws and to the floor joist with hex washer head self-drilling screws. The CFS wall members were connected using 10g square wafer head screws, and the gusset plates were connected using Phillips head screws to the strap braces and chords as they allow a flushed finish.

Three interior walls and seven exterior walls were tested under compression load, which were modified according to the results obtained in each test, as listed in Table 2 and Table 3. The wall/floor/wall specimens were tested on the 300 MTS of the Structures Testing Laboratory of the University of Auckland. The mock-up specimens were cut to the desired lengths to fit into the 300 MTS, as shown in Figure 2 and Figure 3.

Displacement-controlled 1 mm compression loads were applied to the back-to-back chords per 15 seconds using a bracket developed to resist buckling at the point of load application, as shown in Figure 3. The bracket was connected to the chords by 16 mm diameter bolts. A set of M12 8.8 bolts with washers was used to connect the bottom chords and studs to the base of the 300 MTS using unequal angles. The connection was developed to resist any combined horizontal shear and tension forces from the brace action. Figure 3 also shows the location of the two portal gauges, which were used to measure the boundary joists' deformation in the chord and stud areas, and the LVDT, which was used to measure the deformation of the flooring system. A pair of braces was connected to the bottom chords to replicate the effects of the gusset-strap braces connection to the chords and to resist any premature buckling failure.

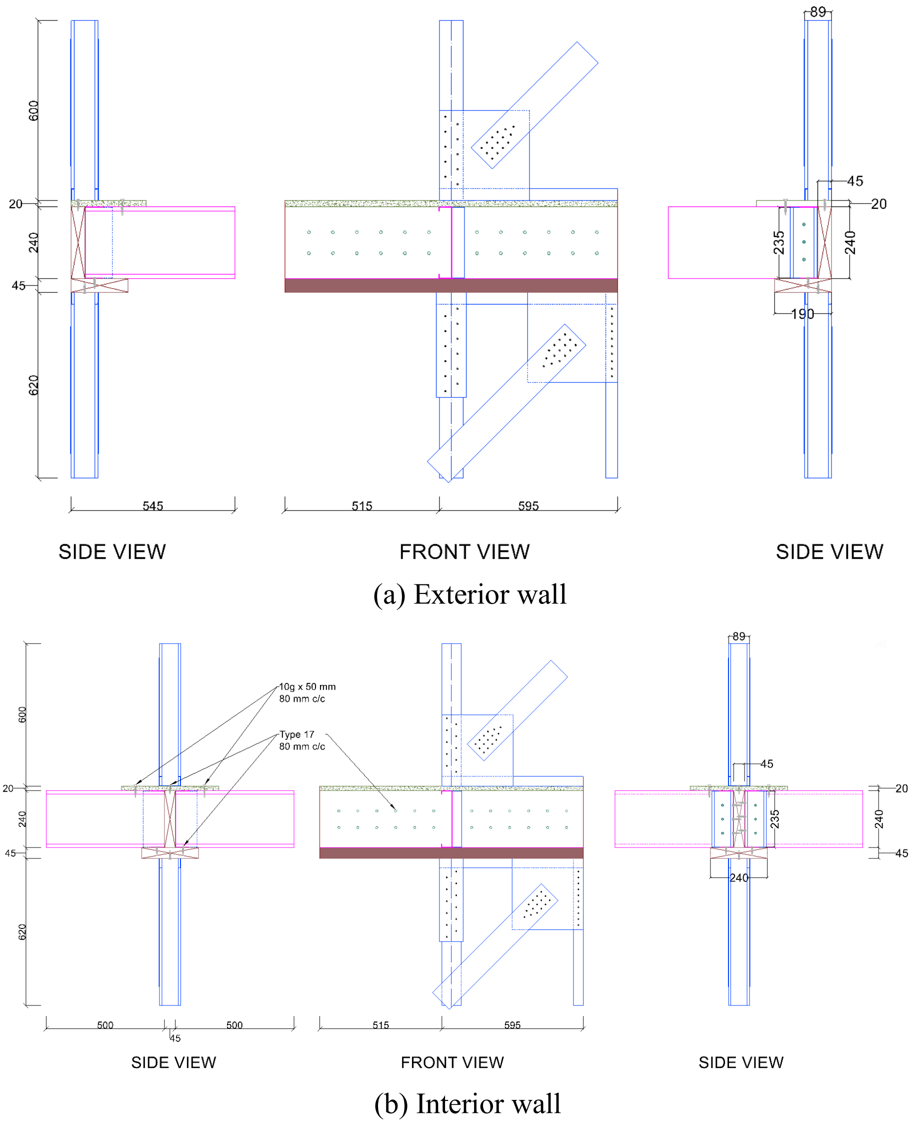


Figure 2: Mock-up of exterior and interior wall/floor/wall test specimen

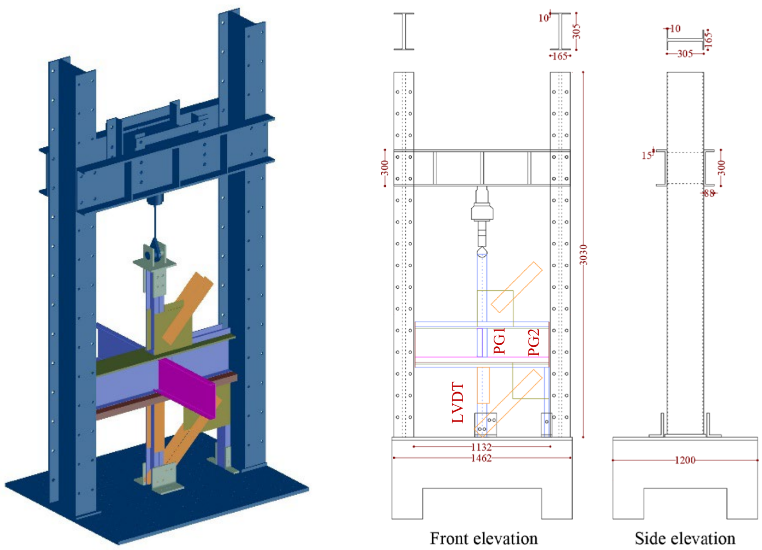
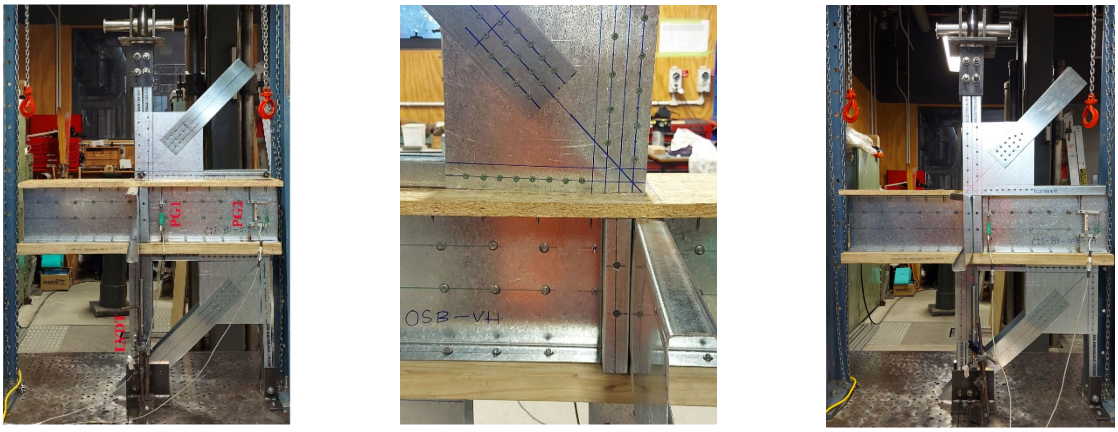


Figure 3 The 300 MTS with a test specimen showing the location of the portal gauges and LVDT



(a) (b) (c)

Figure 4: (a) Test 1, (b) the response of the soft bearing in Test 1, and (c) the strong bearing in Test 2

Table 1: Material properties of the wall/floor/wall components.

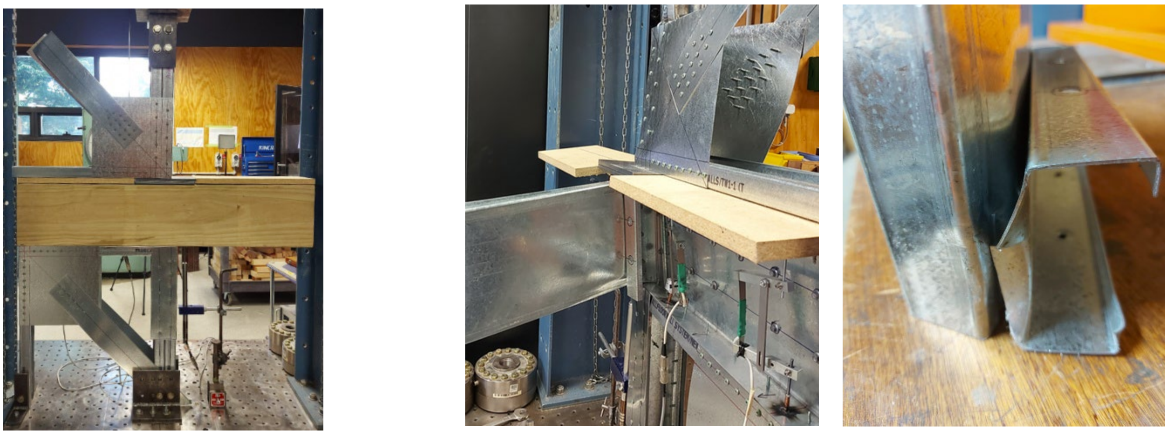
Member (CFS)	Dimension (mm)	Thickness (mm)	Grade	Yield stress (MPa)	Tensile strength (MPa)
Chord	89x40x10	0.95	G500	500	520
Stud	89x40x10	0.95	G500	500	520
Track	89x40x10	0.95	G500	500	520
Strap braces	100	0.95	G250	250	320
Gusset plate	300x300	1.15	G550	550	550
Joist stiffener	89x40x10	1.15	G500	500	520
Floor joist	240x45x15	1.9	G250	250*	280*
Boundary joist	240x60	1.9	G250	250*	280*
Vertical timber	240x45	-	SG8	Compression perpendicular 14 MPa Compression parallel 18 MPa	
Horizontal timber	240x45 190x45	-	SG8	Compression perpendicular 14 MPa Compression parallel 18 MPa	

*Steel Joist System (Speedfloor (2018)

3 RESULTS AND DISCUSSIONS

The key results of the experimental investigation are listed in Table 4. Test 1 was conducted on an interior wall with the configuration of vertical and horizontal timber. At 60 kN load, the compression chord crushed the OSB panel by 2.5 mm, as shown in Figure 4 (b). In a multi-storey building, to increase the building strength, the compression strength of the CFS shear walls needs to be increased by increasing the thicknesses. The thicker chords and tracks will carry higher loads generated from the vertical loading and lateral forces, which will make the OSB panel under the high-compression chords and tracks a soft-bearing region susceptible to crushing failure. Test 1 showed that the crushing failure generated a bending moment in the chords, reducing its load-carrying capacity and thereby yielding at 42 kN load. However, in Test 2, where the OSB panels as a soft-bearing surface were removed, and the tracks were connected directly to the floor joist system, as shown in Figure 4 (c), the yielding capacity of the system increased by about 70%. To add a strong bearing region, a 20 mm steel plate was introduced underneath the back-to-back chords in the remaining seven tests.

The Test 3 configuration of the exterior wall with only vertical timber and a 20 mm steel plate, as shown in Figure 5 (a), resulted in a reduced system strength compared to Test 2 but had an increased strength by about 1.5 times compared to Test 1. However, the steel plates created a stiff load path between the wall panel and the floor joist system. Since only half of the joist stiffener was in bearing with the inside flanges of the boundary joists, the chords compressed the boundary joists' flange, resulting in slipping and bearing failure of the stiffeners, as shown in Figure 5 (b). The result called for a perfect bearing of the joist stiffener with the boundary joists. Joist stiffeners of Tests 4 and 5, shown in Figure 6 (a) and (b), proved that orienting the joist stiffeners with the web of the boundary joist so that their connection with the floor joist is flange-to-web doesn't eliminate the bearing failure. A certain residual displacement was observed due to the gap created between the interface of the boundary joists and stiffeners due to the radius of the boundary joists and facilitating construction. Under the compression load, the system bedded in, and the stiffeners came in hard bearing with the boundary joist. With an increase in load, the crimped end of the floor joist caused the flange of the stiffener to slip and fail in bearing.



(a) (b)

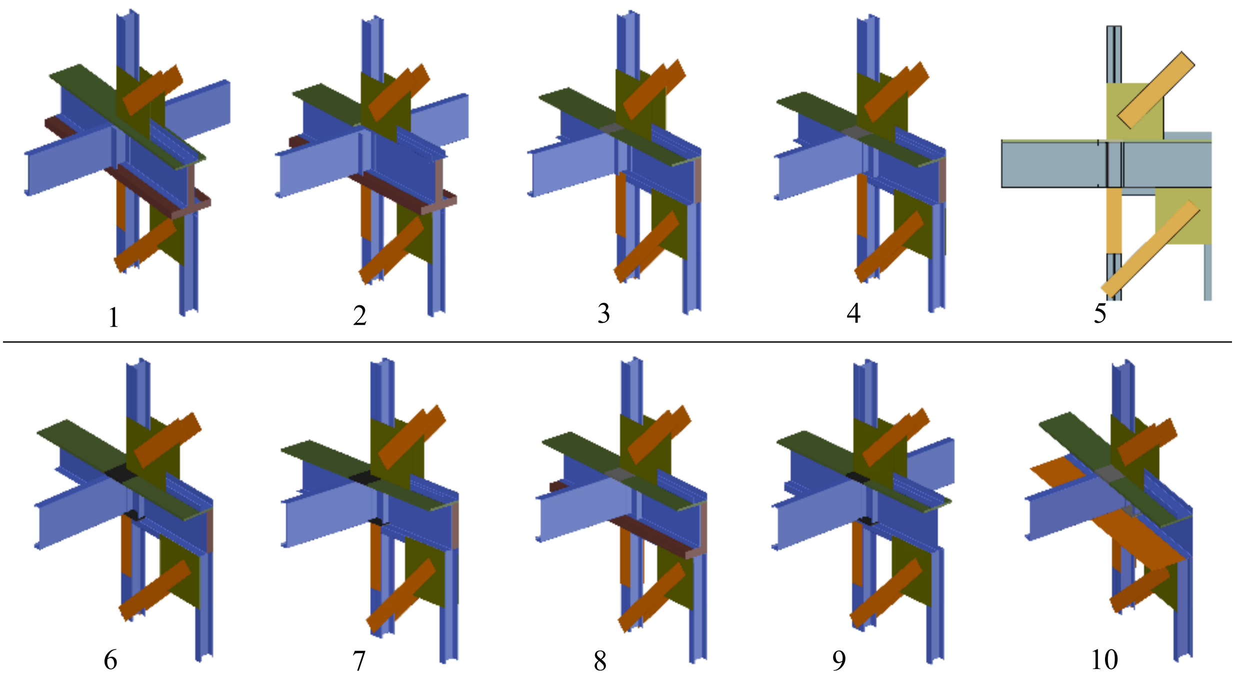
Figure 5: The stiff load path generated by the steel plates underneath the compression chords in Test 3

Table 2: Details of the wall/floor/wall compression test specimens.

Test no.	Wall system	Timber packer	JS to FJ connection	Modification with chords	Modification with joist stiffener	Extra modification
1	Interior	Vertical & horizontal	Web-to-web	-	-	-
2	Interior	Vertical & horizontal	Web-to-web	No flooring replicating a hard bearing under the chord	-	-
3	Exterior	Vertical	Web-to-web	20 mm steel plate under	-	-

4	Exterior	Vertical	Flange-to-web	20 mm steel plate under	-	-
5	Exterior	Vertical	Flange-to-web	20 mm steel plate under	In alignment with chords	-
6	Exterior	Vertical	Flange-to-web	20 mm steel plate under	In alignment with chords, 5 mm bearing plate under	-
7	Exterior	Vertical	Flange-to-web	20 mm steel plate under	In alignment with chords, 10 mm bearing plate under	-
8	Exterior	Vertical & horizontal	Flange-to-web	20 mm steel plate under	In alignment with chords	-
9	Interior	-	Flange-to-web	20 mm steel plate under	In alignment with chords, 5 mm bearing plate under	-
10	Exterior	-	Flange-to-web	20 mm steel plate under	In alignment with chords, 5 mm bearing plate under	Steel plate under floor joist system

Table 3 The tested configuration of all ten specimens



The initial tests showed that the compressive nature of the chords is strong enough to create flange crushing of the boundary joists. In the vertical and horizontal timber configuration of Test 8, the chords of the bottom panel crushed the horizontal timber by 7 mm, as shown in Figure 8 (a). The vertical timber resisted the web crippling, but due to the stiff behaviour of the steel members and the crushing of the horizontal timber, a rotation in the top panel was observed. The rotation caused a moment in the top chords, which led to its failure at 70 kN load. The stiffness of Test 9 northern hemisphere configuration with a 5 mm plate was about twice compared to the stiffness obtained in Test 8. This proved that under the high compression load, the timber packers are susceptible to crushing, thereby reducing the system stiffness and causing the chords to premature failure.

A loading regime with an incremental loading of +5 kN and unloading to zero was adopted for further tests to determine the elastic point of the assembly and the point of change from elastic to inelastic properties of the system. The bearing plates under the joist stiffener in Tests 6 and 7 eliminated the slipping and bearing failure of the joist stiffener. Ideal local buckling of the stiffener in Test 6 was observed at 20 kN, illustrating its engagement with the platform wall/floor/wall system by developing a direct stiff load path, as shown in Figure 6 (c). With a gradual increase in load, distortional buckling was observed in the flanges of the joist stiffener and flange crushing was observed due to the compressive nature of the bottom wall's chords. The load-displacement plots in Figure 7 show that the 10 mm plate in Test 7 increased the stiffness of the assembly by 1.5 times more than in Test 6. To avoid a highly stiff joist assembly interacting with the yielding of the other capacity-designed members under a system behaviour, a 5 mm bearing plate is recommended instead of a 10 mm bearing plate.

(a) (b) (c)

Figure 6: Different failures observed in the joist stiffeners of (a) Test 4, (b) Test 5, and (c) Test 6

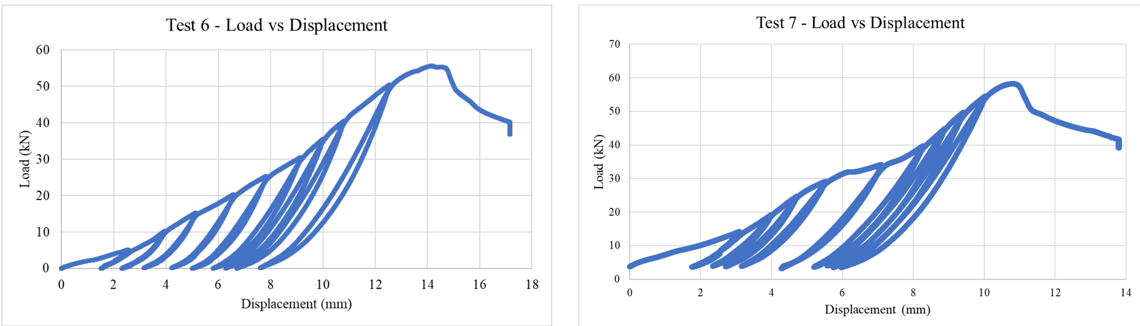


Figure 7: The load vs displacement plots of Test 6 and Test 7

The horizontal timber in the wall/floor/wall system is connected to the ceiling to help resist diaphragm actions in earthquakes and to suppress buckling of the thin gauged steel in severe fires. However, to eliminate the rotation caused by the horizontal timber keeping the fire barrier properties, a G250, 0.95 mm steel sheet was sandwiched between the boundary joist and the top track of the bottom panel in Test 10, as shown in Figure 8 (b). The bottom chord in the compression zone crushed the flanges of the boundary joist, causing the bearing plate to deform. The deformation of the bearing plate at ultimate failure at 57 kN load resulted in the bottom flanges of the joist stiffener failing in bearing. However, the stiffness of the assembly was similar to that of Test 8, indicating that the steel sheet was able to eliminate the rotation that caused eccentric moments in the chords.

(a) (b)

Figure 8: The crushing of the (a) horizontal timber in Test 8 and (b) boundary joists' bottom flange in Test 9

Table 4 The key results of the experimental investigation

Test no.	**Yield point (kN)**	**Stiffness (kN/mm)**	**Test no.**	**Yield point (kN)**	**Stiffness (kN/mm)**
1	42	4.5	2	71	8
3	58	6.5	4	50	7
5	44	4	6	30.5	3.5
7	32	5.2	8	51	4
9	73	7	10	50	3.7

4 CONCLUSIONS AND RECOMMENDATIONS

With the objective of advancing the knowledge of the wall-to-floor-to-wall connection for a CFS shear wall system with platform framing configuration, ten experimental tests were performed on the exterior and interior CFS shear wall system. The tests were done on a 300 MTS under a displacement-controlled loading regime at the University of Auckland.

The key findings from the compression test programme are as follows:

- The initial tests showed that the OSB, under the high compression load carrying back-to-back chords of a strap-braced shear wall in a multi-storey building, created a soft-bearing region, as shown in Figure 4 (b). In a multi-storey building, the strength of the steel system can be increased by incorporating thicker chords and tracks to carry higher loads, but the strength of the flooring material cannot be changed significantly. The results show that the steel plates under the chords prevented the crushing failure of the flooring material and eliminated the soft-bearing region under the high-compression chords, as shown in Figure 5. The steel plate under the chords also led to the joist stiffener's full engagement in transferring the high compression loads by generating a direct stiff load path between the top-storey wall to the bottom-storey wall. However, steel plates will conduct heat in a fire; therefore, a high through-thickness compression capacity passive fire protection board is recommended instead.

- It is essential that the joist stiffeners act as stiffer members so that the load from the top storeys are transferred to the bottom storey wall panels without any bearing failure in the floor joist system. When the joist stiffeners are in a web-to-web connection with the floor joists, the half-bearing with the flanges of the boundary joists tears the web of the stiffeners, as shown in Figure 5 (b). On the other hand, when the joist stiffeners are in a flange-to-web connection with the floor joists, the stiffener slides and fails in bearing after its gap with the boundary joist flanges is closed, as shown in Figure 6.
- The results suggest that a high compression capacity bearing plate under the joist stiffeners helped in closing the gap that the radius of the boundary joists and facilitating construction created, preventing the bearing failure, thereby creating a stiff load path between the walls of the top-storey and bottom-storey.
- In the test specimen with timber packers, it was seen that the compression load in the chords crushed the horizontal timber, as shown in Figure 8 (a). A steel sheet sandwiched between the boundary joists' bottom flange and the top track of the bottom wall is a potential replacement for the horizontal timber. The increased stiffness of the all-steel configuration in an interior wall system proves that the vertical timber is not necessary as the back-to-back boundary joists resist web crippling and any associated failure.

5 REFERENCES

AS/NZS 4600. 2018. *Cold-Formed Steel Structures*. Wellington: Standards New Zealand. www.standards.govt.nz.

Fox, D M, D R Knill, and R M Schuster. 2012. "Load Distribution in Floor to Wall Connections." In *International Specialty Conference on Cold-Formed Steel Structures*, St. Louis, 1–15. https://scholarsmine.mst.edu/isccss/21iccfss/21iccfss-session4/8.

Fox, S R. 2005. *Strength of CFS Floor Assemblies with Clip Angle Bearing Stiffeners*. https://scholarsmine.mst.edu/ccfss-aisi-spec.81.

Fox, Steven R. 2002. "Bearing Stiffeners in Cold-Formed Steel C-Sections." University of Waterloo. https://library-archives.canada.ca/eng/services/services-libraries/theses/Pages/item.aspx?idNumber=55474757 (January 8, 2025).

Fox, Steven R. 2003. *The Strength of Stiffened CFS Floor Joist Assemblies with Offset Loading*.

Fox, Steven R, and G Wayne Brodland. 2004. "Design Expressions Based on a Finite Element Model of a Stiffened Cold-Formed Steel C-Section." *Journal of Structural Engineering* 5(130): 708–14. doi:10.1061/ASCE0733-94452004130:5708.

Hutchinson, Tara C, Xiang Wang, Gilbert Hegemier, Praveen Kamath, and Brian Meacham. 2021. "Earthquake and Postearthquake Fire Testing of a Midrise Cold-Formed Steel-Framed Building. I: Building Response and Physical Damage." *Journal of Structural Engineering* 147(9). doi:10.1061/(ASCE)ST.1943.

Karmakar, A, G C Clifton, and J Bp Lim. 2023. *Numerical Investigation of Australasian Cold-Formed Steel Strap-Braced Walls under Lateral and Vertical Load*. Auckland. https://repo.nzsee.org.nz/xmlui/handle/nzsee/2634 (December 26, 2023).

Lotfi Rad, Pouya, Charles Clifton, James Lim, and Iman Hajirasouliha. 2023. "Experimental Investigation on Seismic Performance of Cold-Formed Steel Strap-Braced Stud Walls under Lateral Cyclic and Vertical Loading." *Thin-Walled Structures* 194: 1–18. doi:10.1016/j.tws.2023.111312.

Natário, P., N. Silvestre, and D. Camotim. 2014. "Computational Modelling of Flange Crushing in Cold-Formed Steel Sections." *Thin-Walled Structures* 84: 393–405. doi:10.1016/j.tws.2014.07.006.

Natário, P., N. Silvestre, and D. Camotim. 2017. "Web Crippling of Beams under ITF Loading: A Novel DSM-Based Design Approach." *Journal of Constructional Steel Research* 128: 812–24. doi:10.1016/j.jcsr.2016.10.011.

Speedfloor. 2018. *Steel Joist System Product Manual*.

Wang, Xiang, and Tara C. Hutchinson. 2021. "Earthquake and Postearthquake Fire Testing of a Midrise Cold-Formed Steel-Framed Building. II: Shear Wall Behavior and Design Implications." *Journal of Structural Engineering* 147(9). doi:10.1061/(asce)st.1943-541x.0003098.