
From theory to test: seismic and gravity loading insights into platform-framed cold-formed steel structures via review and full-scale experiments

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ABSTRACT

The growing demand for increased population density in major cities of New Zealand is increasing the need for medium-rise residential buildings (around six storeys). The high strength-to-width ratio, capabilities of modular construction and seismic resiliency of cold-formed steel (CFS) structures present a potential new, cost-effective solution to meet the growing demands. The industry prefers the platform framing systems as they facilitate the effective construction of one-storey at a time, which is standard for two-storey buildings, and they have enhanced stability under severe fire conditions. However, for a potential development, a deeper understanding of platform framing's multi-storey system behaviour under high seismic and vertical loading is required. This paper investigates the seismic and vertical loading performance of platform-framed multi-storey CFS structures, bridging theoretical understanding with full-scale experimental tests that have recently been completed at the University of Auckland. Insights from a comprehensive literature review of design assumptions, wall/floor/wall connections, and energy dissipation of strap-braced shear walls formed an integral part of the two-storey experimental programme. The full-scale tests have confirmed the adequacy of the proposed lateral and vertical load transfer mechanism through the top storey wall to the flooring system, the flooring system to the bottom storey and finally to the foundation system. The key failure patterns have been summarised, which provides a valuable insight into the seismic and vertical design criteria, demonstrating the importance of combining theoretical review with experimental validation. The outcome of this study, when implemented, will significantly advance the application of light-framed modular construction for medium-rise residential buildings.

1 INTRODUCTION

Cold-formed steel (CFS) systems offer advantages in terms of lightweight, ease and efficient construction, low embodied carbon, high-dimensional accuracy, resistance to warping, shrinking and creeping under load. Cold-

formed steel members' enhanced structural efficiencies with regard to increased buckling and crippling strength are well-suited to system development. When CFS is incorporated into a seismic-force resisting system (SFRS), strap-braced shear walls and steel sheathed shear walls are the most common forms of lateral force resisting system (LFRS). Despite the structural advantages and the ongoing research into enhancing the seismic performance of such shear walls, understanding the system-level performance of the shear walls under realistic conditions with flooring systems remains limited, especially for platform construction. The lack of extensive experimental data constitutes a major shortcoming in the knowledge required to understand the behaviour of light-framed multi-storey buildings in high-seismic countries like New Zealand.

The construction of CFS systems uses the modular form of construction, where the prefabricated CFS assemblies (wall panel systems, flooring systems and roofing systems) are brought on-site and connected, thereby facilitating faster construction. Since CFS is highly recyclable and its thinner section profiles allow greater insulation depth, it reduces operational energy demand and is therefore regarded as an eco-friendly construction material. The platform framing, ledger framing and balloon framing are the three types of framing systems that are employed in modular construction, amongst which platform framing is preferred by the New Zealand CFS industry. The *balloon framing* came into existence in the mid-1800s, which was typically used in the single-to-two-storey standalone houses (Isaacs 2013), where the studs ran throughout the height of the two-storey building. Gradually, *ledger framing* became popular in the industry, where the floor joist is connected to the inside face of the wall studs using a joist end track (also called a ledger track) and joist stiffener clips, as shown in Figure 1. The floor sheathing is sandwiched between the tracks of the top and bottom walls, and the loads are transferred through direct bearing of the wall tracks through the joist stiffener clips. In a severe fire, the ledger clip is only protected by the ceiling linings, and movement of the ledger clip away from the wall is likely, which poses a high risk of failure in a severe fire.

In contrast, with *platform framing*, the floor joist systems are connected to the top track of the bottom wall, and then, with the floor sheathing, the bottom track of the top wall is connected in alignment with the wall studs, as shown in Figure 2. The platform framing also has a high tolerance for placing the flooring systems on top of the load-bearing walls. The disadvantage compared with ledger framing is the requirement to transfer axial forces from the upper wall to the lower wall through the flooring system, without causing crushing or other forms of failure within the floor joist system.

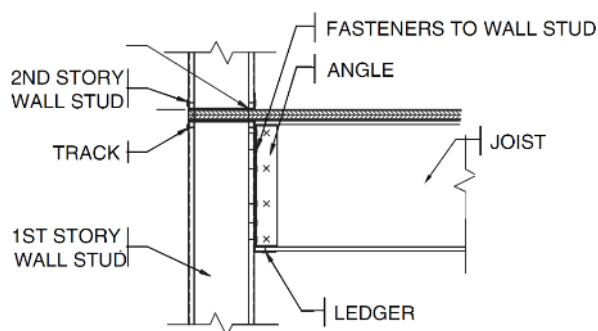


Figure 1: The ledger framing flooring system of the CFS-NEES full-scale building (Peterman et al. 2016)

1.1 Background

In a global multi-storey building system, with the cyclic nature of the earthquake forces, large compression forces are transferred from the top storey shear walls to the bottom storey walls through the flooring system, due to the combined earthquake and gravity system loads. These compression forces are transferred through the back-to-back chords, which are significantly higher than the tension forces generated from the earthquake forces; however, the tension forces also require consideration. This necessitates a strong and stiff load path from the upper walls through the flooring system and into the lower walls.

Some attempts have been made to investigate this load transfer mechanism at the system level through shake-table experimental tests on low-rise buildings or scaled versions of medium-rise buildings. However, most of these studies considered steel sheathing, oriented-strand board (OSB) sheathing, or gypsum board sheathing as the shear walls (Leng et al. 2017; Leng, Buonopane, and Schafer 2020; Peterman, Madsen, and Schafer 2014; Zhou et al. 2022). Another detailed multi-hazard study of a six-storey steel sheathed building by Hutchinson et al. (2021), Wang and Hutchinson (2018, 2020, 2021) considered ledger framing. Other work

includes three-storey prototype shake-table tests on strap-braced walls, but with steel-concrete and wood-based flooring systems (Campiche et al. 2019; Fiorino, Bucciero, and Landolfo 2019; Landolfo et al. 2021; Shakeel et al. 2019; Terracciano et al. 2018).

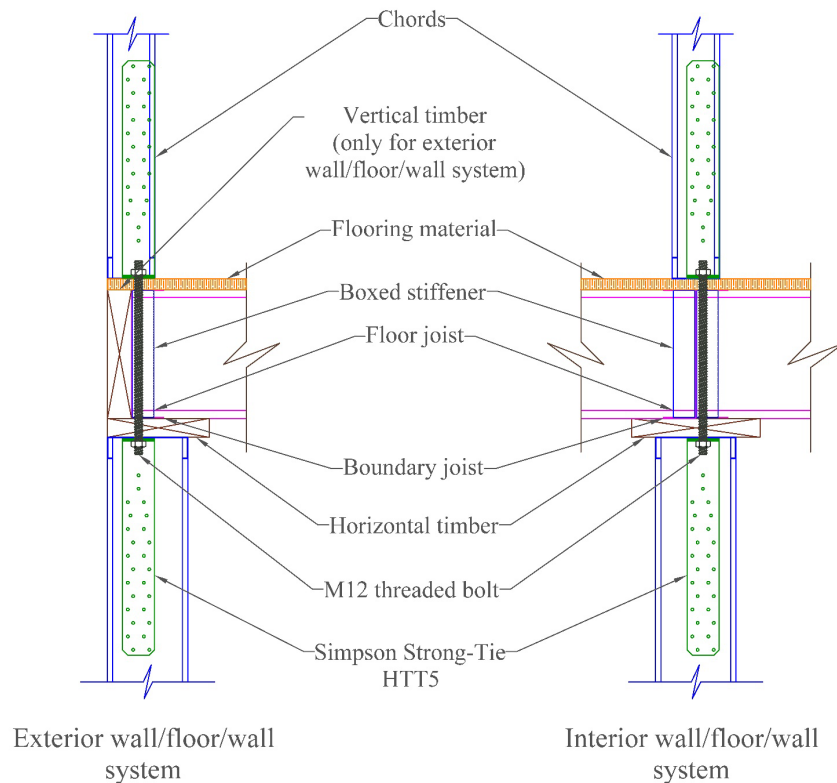


Figure 2: The proposed platform framing system for the exterior and interior wall/floor/wall system

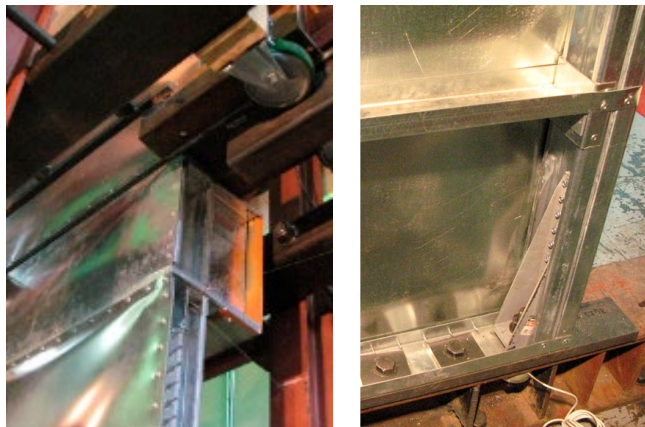


Figure 3: The two-storey platform-framed steel sheathed CFS shear wall study images by Shamim, Dabreo, and Rogers (2013)

A related CFS platform framed study by Shamim, Dabreo, Rogers (2013), Shamim and Rogers (2013) investigated the lateral performance of 2:1 aspect ratio steel sheathed shear walls. The configuration incorporated the Simpson Strong-Tie S/HD 10S hold-down system on the inside face of the chords along with joist stiffeners between the flanges of the floor joists, as shown in Figure 3. An innovative technique to connect the nogs with the chords was incorporated using blockings. While these details enhanced the performance of the steel sheathed shear walls with platform framing, such systems inherently exhibit more complex behaviour than strap-braced systems due to fastener slip, screw tear-out and panel-frame interaction.

The strap-braced system offers a simpler tension load path and avoids the installation complexity due to higher screw quantities and increased material handling associated with steel sheathing. Finally, all these tests have used medium-strength, thicker studs compared with the very high-strength, thin-gauge studs used in New Zealand CFS construction.

As can be observed, comprehensive experimental data on the combined lateral and vertical load-carrying performance of strap-braced shear walls with platform framing remain limited. Capturing the complex

interactions between the strap-braced shear walls, the platform framing system, and the connections between them is essential in determining the system-level performance of multi-storey CFS strap-braced platform framed buildings constructed from thin-gauge, high-strength steels in seismic areas. To address this gap, the present study reports six full-scale two-storey CFS strap-braced wall/floor/wall experiments designed to observe the lateral and vertical load-carrying capacity of a platform-framed system. This paper is intentionally written for industry practitioners, with the primary aim of communication that the most anticipated year-long full-scale two-storey CFS strap-braced platform-framed experimental tests have been completed on 22 January 2026 at the University of Auckland. Additionally, the study presents the visual observations and insights from the overall wall/floor/wall behaviour, providing a system-level understanding for practical design and construction use.

2 THE FULL-SCALE TEST DETAILS

2.1 Material properties and test matrix

The material properties of all the members conformed to AS/NZS 4600 (2018). The wall members were made of G550 steel for 0.95 mm thick studs and G500 steel for 1.15 mm. The gusset plates were G500, and the thicknesses remained 1.15 mm for all walls. Since the strap braces were designed to yield, they were of grade G250. The Speedfloor (2018) joist system was used for the lipped C-section floor joist and U-section boundary and perimeter joist, which were graded G250. Plywood sheathing of 20 mm thickness was used for the flooring material. The Simpson Strong-Tie's HTT5 holdown system was used for the anchorage connecting the top wall to the flooring system and the bottom wall with an M12 4.6/S category threaded bolt.

Five tests were undertaken with an 89 mm wide top shear wall and a 150 mm wide bottom shear wall, and one test with 150 mm wide top and bottom shear walls, as listed in Table 1. Among the six full-scale two-storey tests, two test specimens were set up for the exterior wall/floor system and four for the interior wall/floor system. To capture the difference in the lateral performance of the shear wall, different dimensions and configurations of the strap braces were adopted, which resulted in a significantly different wall-to-floor-to-wall response for each test configuration.

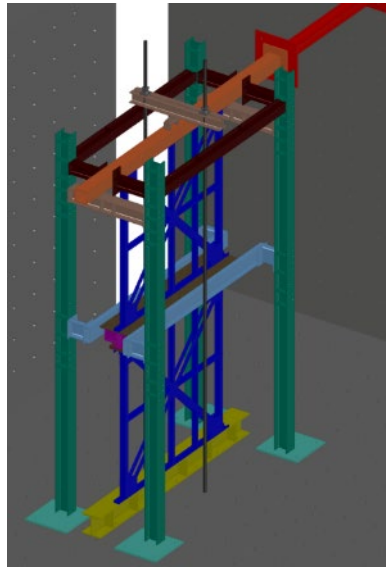
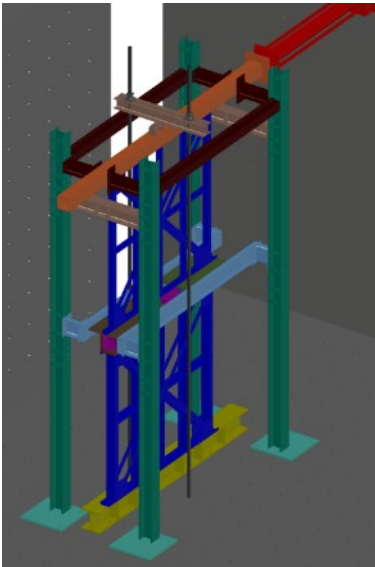
Table 1: The test matrix for full-scale two-storey platform framed wall/floor/wall system.

Test	Wall	W/F system	Straps	Test	Wall	W/F system	Straps
1	89 – top	Exterior	80 – both side	2	89 – top	Interior	80- both side
	150 – bottom		100 – both side		150 – bottom		100 – both side
3	89 – top	Exterior	160 – one side	4	89 – top	Interior	160 – one side
	150 – bottom		200 – one side		150 – bottom		200 – one side
5	89 – top	Exterior	125 – both side	6	150 – top	Exterior	125 – both side
	150 – bottom		125 – both side		150 – bottom		125 – both side

2.2 Loading protocol

The loading protocol adopted for the cyclic loading was similar to the Method B of ASTM E2126 (2019). However, to accommodate the two-storey into the predefined loading protocol of Method B, minor changes were made based on the total height of the two-storey specimen. In addition, the design methodology considered the bottom wall to be 1.25 times stronger than the top wall, as part of the system to distribute

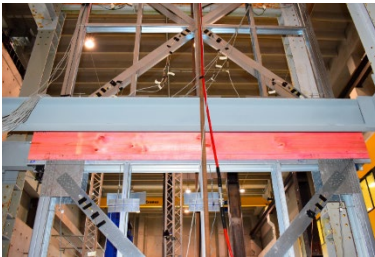
seismic demand reasonably uniformly up the building. The aim of adopting this approach was that, in a real multi-storey building, the bottom storey walls are required to be seismically stronger than the top storey walls.



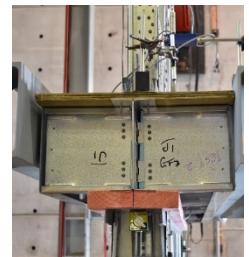
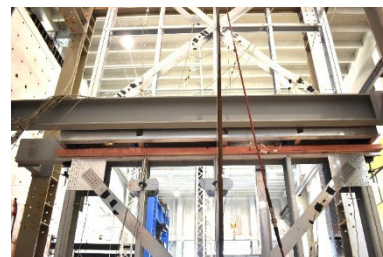
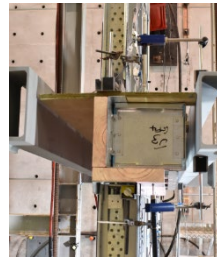
(a) 3D illustration of the interior and exterior system



(b) Test 1 (before test)



(c) Exterior wall system



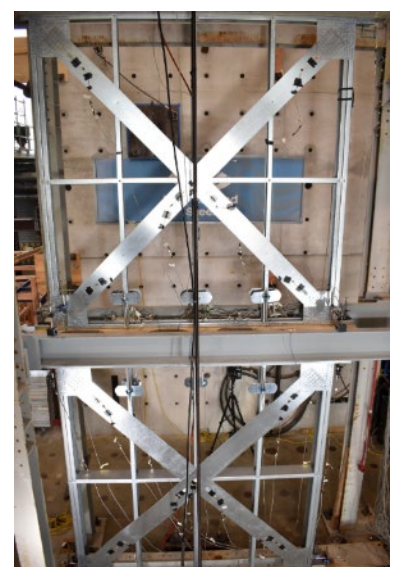
(d) Interior wall system



(e) Test 2 (before test)



(f) Test 3 (before test)



(g) Test 4 (before test)



(h) Test 5 (before test)



(i) Test 6 (before test)

Figure 4: The different visuals from the full-scale two-storey experimental programme

This method deliberately forced the top walls whose top tracks were bolted to the loading beam into the inelastic range, thereby keeping the bottom walls and the flooring system elastic. It was recorded that the self-weight of the actuator at Level 2 was partially taken by the back-to-back chord near the actuator and the first stud from the right. The Case 1 vertical loading only considered the self-weight of the actuator and was applied from 0.05% to 1.5% drift ratio. Whereas Cases 2 and 3 of vertical loading of 20 kN and 30 kN, were applied at 1.75% and 3% drift ratios, respectively. These vertical loads corresponded to the design calculations undertaken for a case study vertical load-bearing wall for a four and six-storey building.

3 INITIAL OBSERVATIONS

The six full-scale two-storey platform framed wall/floor/wall experimental tests were conducted at the Structures Testing Laboratory of the University of Auckland. Being completed on 22 January 2026, limited data processing has been undertaken at the time of writing this paper. Therefore, this paper focuses on documenting the observed behaviour and key qualitative findings through visual observations. The intention is to keep the wider industry informed of the progress of this research and the system-level insights that are sought to emerge from the testing. Figure 4 shows images from the various wall/floor/wall systems before the tests.

- Test 1: The system retained its self-centring capabilities at a 1.75% drift ratio (42 mm displacement). There were no significant deformations and negligible floor crushing at 20 kN, which was applied at a 2.75% drift ratio and at 30 kN, which was applied at 4% drift ratio. The system lost its vertical load-carrying capacity when a combined axial compression and bending moment about the x-axis caused the back-to-back chords near the Big Red Actuator to fail. The failure occurred at the negative cycle of the 5% drift ratio when the chord under the actuator was at the maximum compression load generated by the lateral and vertical loading, as shown in Figure 5 (a).
- Test 2: The system showed no significant deformation at the 2.5% drift ratio. However, the ultimate failure occurred at 4% drift ratio when the chords under the actuator, which carried a 5.5 kN load from the self-weight of the actuator in all the specimens, failed in combined axial and bending action, as shown in Figure 5 (b). An in-plane double curvature resulting from the frame action was also observed. At the end of the 4% drift ratio, the lateral actuator was centred, and vertical load was applied to record

the gravity load-carrying capacity at that yield level. It was observed that the specimen resisted a 75 kN vertical load without any significant crushing of the flooring system.

- Test 3: For this exterior wall system test specimen, the vertical load applied at 1.75% drift was different from the nominal loading protocol. A pressure set point error resulted in a transient axial load of approximately 190 kN. This elevated load was applied for 14 seconds. The vertical load was immediately stopped, and the pressure machine was recalibrated. Following the correction, the previous defined 20 kN vertical load was applied, and the experiment resumed according to the specified loading sequence. Before reaching the complete positive cycle of 4% drift ratio (96.8 mm displacement), large tension forces were transferred through the strap braces, which travelled through the gusset plates to the bottom track of the top wall and into the gusset-chord-track region. Due to the space constraint that gusset plates create at the chord-track junction, the top wall was not connected to the flooring system with sufficient screws, which resulted in the web's tensile fracture, as shown in Figure 5 (c). However, it should be noted that no significant damage was observed at the ultimate limit state drift ratio of 2.5% without any floor crushing.
- Test 4: A similar failure response to that of Test 3 was obtained, where, before reaching the complete positive cycle of 4% drift ratio, the huge tension forces caused a net section fracture in the tracks, as shown in the Figure 5 (d). This suggests that the one-sided strap-braced walls should be connected to the flooring system with densified screws beyond the region of the gusset plates for design considerations beyond the ultimate limit state. After the net section failure, the lateral actuator was centred, and an incremental vertical load was applied to the specimen. It was observed that after a 60 kN vertical load, the wall/floor/wall system could not carry any further load.
- Test 5: No significant deformation was observed at 2.75% and 3% drift ratio. However, beyond the 73 mm displacement in the positive cycle of 4% drift ratio, the local buckling of the outside stud of the back-to-back chord generated an overall global buckling, as shown in Figure 5 (e). Significant tension uplift was seen on the bottom walls connected to the foundation beam compared to the previous test specimens. The 4.6 category M12 bolts connecting the wall to the foundation beam showed in-plane deformation. In the negative cycle of the same drift ratio, the bolts connecting the left chord to the foundation beam through Simpson Strong-Tie HTT5 holdown fractured.
- Test 6: A similar in-plane deformation to that of Test 5 was observed in the 4.6 category M12 bolts at the 2.75% drift ratio. At 3% drift ratio, the chords under the lateral actuator showed compression failure, as shown in Figure 5 (f). Therefore, the lateral actuator was centred after the completion of the 2.75% drift ratio, and the vertical load was applied incrementally. The specimen showed no compression failure at 50 kN load. This suggests that a Property Class 8.8 M12 bolt should be used for all the anchorage systems from the bottom wall into the foundations and possibly for some of the lower wall/floor/wall junctions. These PC 8.8 M12 bolts provide an axial tension capacity of 54 kN, rather than 27 kN for PC 4.6 category M12 bolts.

4 CONCLUSION

The six full-scale two-storey platform-framed wall/floor/wall generally exhibited a stable behaviour up to approximately 2.5% - 3% drift ratio, after which failure mechanisms such as the chord's axial – moment failure, connection fractures, and chord buckling were observed. Tests 1 and 2 demonstrated self-centring behaviour at low drifts. Tests 3 and 4 showed failures around gusset plate regions where high-tension forces from the strap braces led to premature web and track failures, while Tests 5 and 6 demonstrated the limitations of PC 4.6 M12 bolts under tension uplift and compression demands. In summary, the test results emphasise the influence of drift levels on system performance and the need for enhanced connections and anchorage capacity in a platform-framed CFS multi-storey building system.

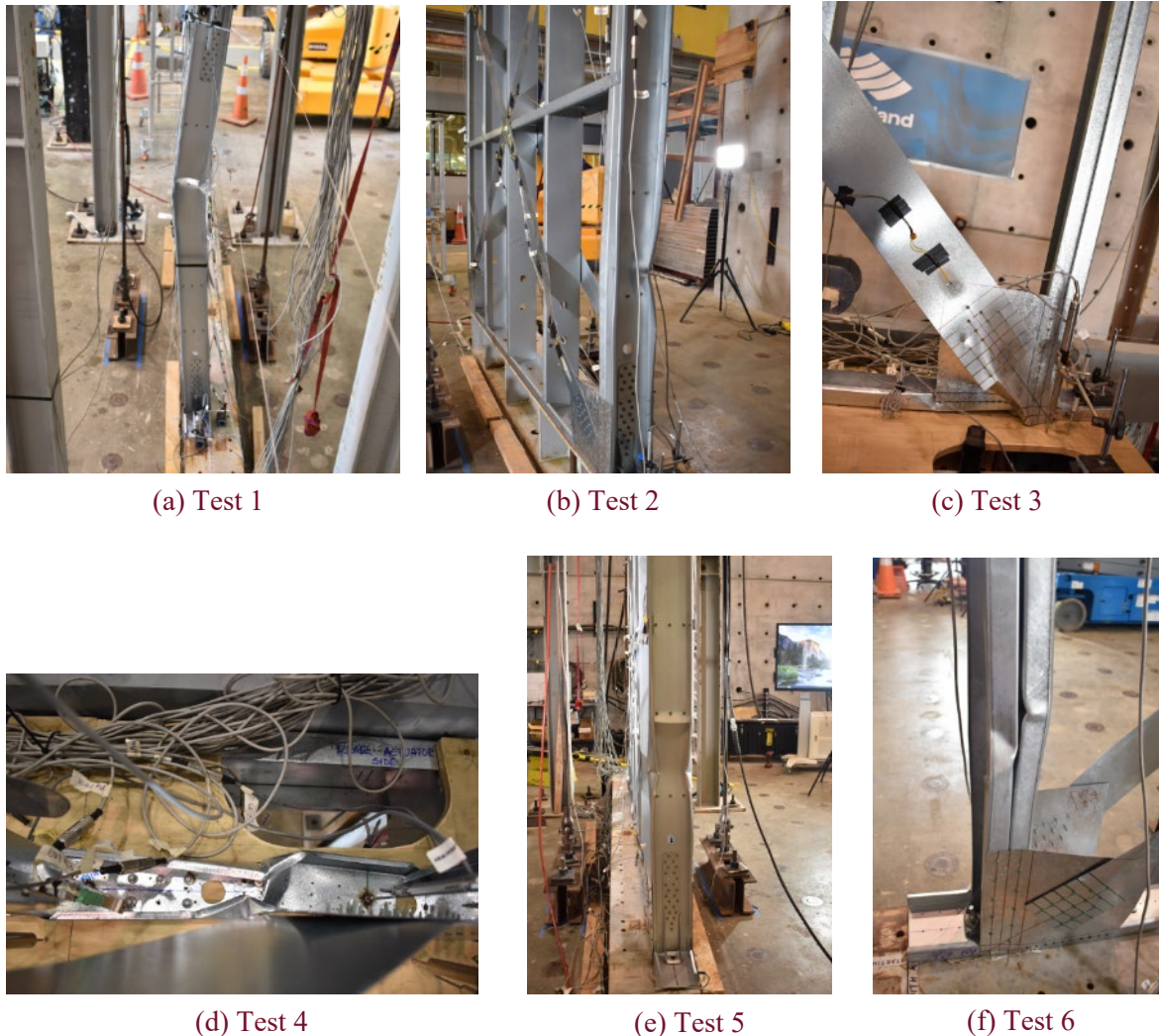


Figure 5: The different major failure modes obtained after the end of each test

5 ACKNOWLEDGEMENTS

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