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Analytical Investigation of a Novel Joist-to-Stud Web Connection in Light Steel Framed Buildings

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Abstract

In typical light steel framed (LSF) buildings, floor joists are connected to studs through web connections assuming pinned behaviour. This often results in deeper joist sections, as the design is primarily governed by mid-span deflections. Consequently, the full load-bearing capacity of cold-formed steel (CFS) elements is underutilized, leading to heavier structures and increased environmental impact. This paper investigates the behaviour of a novel semi-rigid joist-to-stud connection, where the joist and stud webs are screwed together. The semi-rigid nature of this connection allows for the development of rotational stiffness and bending resistance, enabling the use of smaller joist sections and more efficient utilisation of structural capacity. To this end, detailed experimentally validated Finite Element (FE) models are developed in ABAQUS software to assess the influence of key design parameters, including connecting element sizes, screw arrangements, construction methods and gravity loads, on the structural performance of joist-to-stud connections. The performance of the connections is compared in terms of initial stiffness and flexural strength. Depending on the screw configurations and the section sizes, two main failure mechanisms are anticipated: (i) shear failure in the screwed connection; (ii) local buckling of the stud or joist flanges near the connection zone. The results indicate that implementing a semi-rigid connection led to an average 25% reduction in the steel weight of the structure of six storey case study buildings compared to its conventionally designed counterpart with simple connections.

Keywords

finite element, flexural strength, joist-to-stud floor connections, semi-rigid connections, cold formed steel

1 Introduction

Light steel framed construction utilizes Cold Formed Steel (CFS) components for walls and floors. The walls consist of vertical load-bearing studs connected to a track, while the floors consist of horizontal load-bearing joists connected to a ledger track. There are three primary methods for assembling walls and floors: platform, balloon, and ledger framing [1]. These methods fall into Sequential or Continuous Construction Methods (SCM or CCM) categories, depending on the continuity of studs between levels. Platform framing constructs one level at a time, creating non-continuous wall studs. Balloon framing maintains continuous wall studs over one storey, with floors suspended from walls using a ledger track. Ledger framing is a hybrid, constructing walls one level at a time but suspending floor joists from walls using a ledger track. In all approaches, floor joists connect to a ledger track or zed section, which in turn connects to wall studs via clip angles. This connection, categorized as a simple shear connection, allows the transfer of bearing forces on studs, with joist design primarily controlled by mid-span deflections under the serviceability limit state [2].

Ayhan and Schafer [3] explored the moment-rotation response of joist-to-stud connections in ledger framing through full-scale experiments. While the connection behaved similarly to a "simple" connection with sufficient rotation capacity, undesirable limit states were noted including stud web crippling and ledger bottom flange at large rotations. Sabbagh and Torabian [4] proposed a novel semi-rigid joist-to-stud connection, eliminating the need for ledger track and clip angles by screwing the joist web directly to the stud web. The semi-rigidity of the connection enables the utilization of lighter joist sections by mitigating mid-span deflections, a factor typically governing the design of joist elements.

This study employs Finite Element (FE) modelling to investigate the behaviour of the novel semi-rigid joist-to-

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stud connection proposed by Sabbagh and Torabian [4] under various influential parameters, such as joist and stud sizes, screw configurations, and gravity loading. The web planar screwed connection is examined for stiffness, moment capacity, and governing failure mechanisms. Additionally, a case study design of a Light Steel Framed (LSF) building demonstrates the potential advantages of the novel semi-rigid connection over currently used simple shear connections.

2 Description of FE model

Finite Element (FE) models were created for a novel semi-rigid connection as illustrated in Figure 1. These models were developed in the ABAQUS [5] software package incorporating considerations for geometric imperfections and material non-linearity.

2.1 Description of FE model

The FE model for the semi-rigid connection encompasses a screwed web connection between the lipped channel sections of joists and studs. A hard surface-to-surface contact was defined for all elements. The stud height, modelled as 2.7m, included a restraint in the out-of-plane direction X (bridging) at mid-height using reference point RP-4 for SCM assemblies as shown in Figure 1. For CCM, the connection was positioned at the mid-height of the stud. A hinged boundary condition was implemented at the bottom of studs using reference point (RP-2) located at middle of two studs to which all degrees of freedom of the stud end section were coupled. Gravity load from upper levels was applied at point (RP-3) located in the middle of studs, coupled to the studs at the top, with free translation and rotation in the vertical (Y-direction) and X-direction, respectively. Translation in the X direction was restrained at the connection location and sides of the sheathing, replicating practical restraints for floor connections in Cold Formed Steel (CFS) buildings.

The joist length of 3.3m was used in the models, representing half the length of floor joists in the CFS NEES building [6]. This choice aligns with the experimental results [3] by Ayhan and Schafer used for validating FE model parameters. Cross-sectional imperfections were also incorporated into the model. A two-step loading process was employed. In step 1, the gravity load was applied at RP-3. In step 2, the model underwent analysis with static displacement imposed at the cantilevered end (RP-1 in Figure 1) of the joist to induce moments in the joist-to-stud connection.

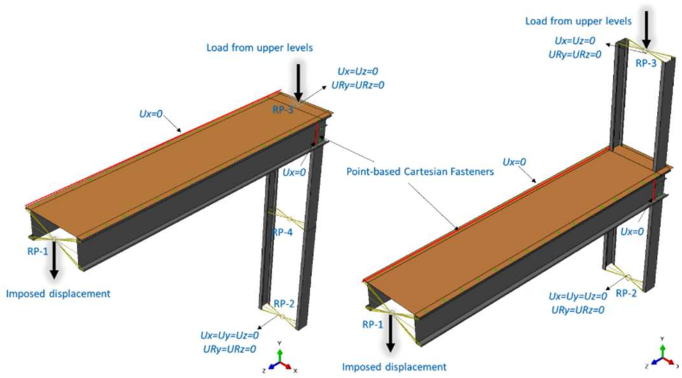


Figure 1 FE model assembly for the semi rigid connection in SCM (left) and CCM (right)

2.2 Element Type and Material Properties

The modelling of steel joists, face tracks, stud channel sections, and OSB sheathing involves the use of shell elements. Following a mesh sensitivity analysis, a suitable mesh size of approximately 10x10 mm^2 was employed for all elements. The steel is modelled by a bi-linear stress-strain curve with a nominal yielding strength of 345 N/mm^2, modulus of elasticity of 203,500 MPa, and a strain hardening ratio 0.01. To provide necessary restraint to joist top flanges, OSB floor sheathing with a thickness of 14.9 mm and modulus of elasticity of 700 N/mm^2 is also modelled atop the joists.

2.3 Modelling of the Screws

The connection between the joist and stud web employs #12 self-drilling screws with a thread diameter of 5.4 mm. OSB sheathing is also connected using #12 screws to the top flanges of joists and the face track. Abaqus Point-based Cartesian Fasteners are used to model the screw connections with a radius of influence equal to the thread diameter of screws. This modelling technique, successfully applied in the FE modelling of Cold Formed Steel (CFS) connections [7], employs quad-linear load-deformation backbone curves for steel-to-steel and OSB-to-steel screw fasteners. These backbones are selected based on the work of Tao et al. [8], containing testing of screwed connections between steel sheathing of different thickness groups, including analytical formulations for predicting backbone curves.

More details on modelling assumptions can be found in [4]. FE results on the proposed novel connection are further discussed in section 4.

2.4 Validation

As the models created here pertain to a new connection assembly yet to be tested, FE validation of modelling assumptions was conducted using available experimental results in the literature for ledger framed connections [3]. FE models were developed for a ledger connection involving a single 1575 mm long joist connected to a ledger track with a 38 × 38 × 1.4 mm clip angle. The ledger track is supported by two studs, each 813 mm high and 600 mm apart. Figure 2 illustrates the test specimen for ledger framed connections tested at John Hopkins University [3].

Steel with a yield stress of 345 N/mm^2 was used for the joist, stud, ledger, and top track sections. These sections were represented by 1200S250-97 (304x64x15.9mm, t=2.5mm), 600S162-54 (152x41x12.7mm, t=1.4mm), 1200T200-97 (304x64mm, t=2.5mm), and 600T162-54 (152x4mm, t=1.4mm), respectively. OSB sheathing was affixed to the joist flange and the wall top track web. Simpson self-drilling #10 screws with a 4.7 mm thread diameter secured every connection. The load deformation behaviour of screwed connections was derived from [8].

Figure 2 illustrates the dominant failure limit state of ledger flange buckling (LFB), consistent with both the FE model and the corresponding test (specimen T4 [3]). Moreover, as depicted in Figure 2, the overall trend of moment-rotation behaviour estimated by FE analysis aligns well with the test results. Predictions for peak strength and initial stiffness from FE analysis are within 5% and 10% range of the test results, respectively. Additional details on FE validation are available in [4].

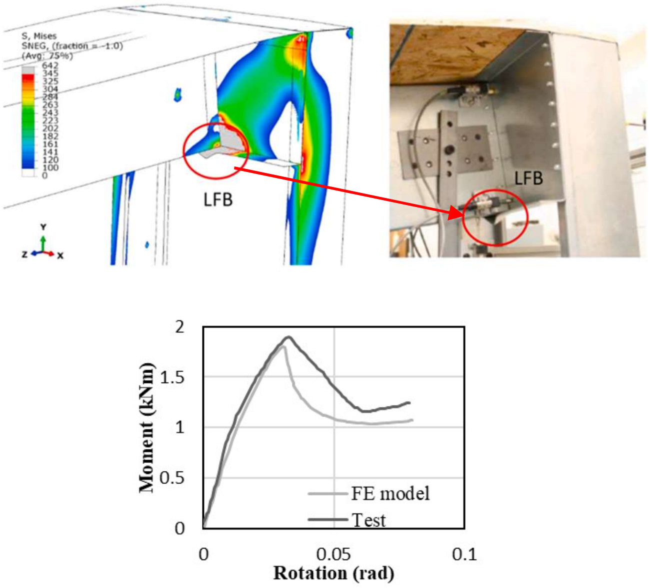


Figure 2 Local Flang Buckling (LFB) observed in FE model for specimen T4 and Leger-framed connection test on specimen T4 [3]; Comparison of moment rotation responses.

3 Parameters under investigation

Following the validation of modelling assumptions as outlined in section 2.4, an investigation of the impact of various parameters on the response of the proposed connection was conducted. The primary objective of this parametric study was to observe different failure mechanisms and their associated moment-rotation responses in the proposed connection. The investigated parameters encompassed gravity loading from upper floor levels, screw arrangement on the connection, combinations of stud and joist thicknesses, and the type of construction method, as elaborated in the subsequent sub-sections.

3.1 Gravity Loading

Gravity loading from upper levels was applied to the studs to examine its effect on the connection response. Three different intensities of gravity loading were considered: 0%, 20%, and 40%. The 0% level denotes the absence of gravity load, while the 20% and 40% gravity loads are computed based on the 20% and 40% load-bearing capacity of studs. This assumption aligns with the typical design scenario where gravity load-bearing studs experience proportion of loads ranging from 20% to 40% of their capacities coming from upper level. It should be noted here.

3.2 Screw Arrangement

Various planar screw arrangements were adopted to connect the webs of studs and joists. Specifically, 1 to 4 vertical lines of screws were utilized. Within each line, 3 rows of screws were employed. The horizontal distances of the outermost lines of screws to the centre of gravity of the screw group were kept constant in the case of 2 to 4 lines of screw arrangements. All cases met the spacing requirement outlined in EN 1993-1-3 [9].

3.3 Joist and Stud Thicknesses

Different combinations of cross-sectional thicknesses for studs and joists were also investigated. Two levels of plate slenderness were used in the joists, while studs had three different slenderness levels. The remaining dimensions for studs and joists, including cross-section depth, flange widths, lip sizes, and longitudinal dimensions, were kept constant.

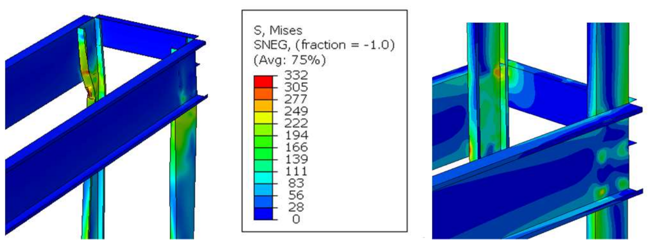
3.4 Construction Method

The impact of employing the proposed novel connection in both Sequential Construction Methods (SCM) and Continuous Construction Methods (CCM) was also explored. Identical connection assemblies were adopted for both SCM and CCM configurations, as illustrated in Figure 1.

Each modelled connection assembly is assigned an identifier tag based on the following nomenclature: The first letter represents the type of construction method (S for Sequential or C for Continuous). The first two numbers represent the thickness of the joist in one ten-thousandth of meters. The subsequent two numbers represent the thickness of the stud in one ten-thousandth of meters. A number followed by a dash (-) represents the number of lines of screws, and the number in decimal fraction followed by the second dash (-) represents the percentage of gravity load (for 0%, this number is not specified).

4 Results and Discussion

The outcomes of the Finite Element (FE) analysis were primarily scrutinized based on moment-rotation responses at the connection and the governing failure mechanisms. The analysis identified two primary failure modes: local buckling of the studs or failure of the joist-to-stud fastener, as depicted in Figure 3. Joist yielding or buckling was not observed in any of the analysed cases.



a) Stud failure b) Joist to stud fastener failure

Figure 3 Failure mechanisms (Floor sheathing removed for sake of clarity)

The subsequent set of figures presents moment-rotation curves for various connection assemblies, revealing two distinct curve shapes. Connections lacking a post-peak response are influenced by the joist-to-studs connection failure, while those with a substantial post-peak response, truncated to a 20% drop after the peak moment, are governed by stud failure. All curves exhibit a consistent pre-peak pattern characterized by a bilinear slope leading up to the peak point.

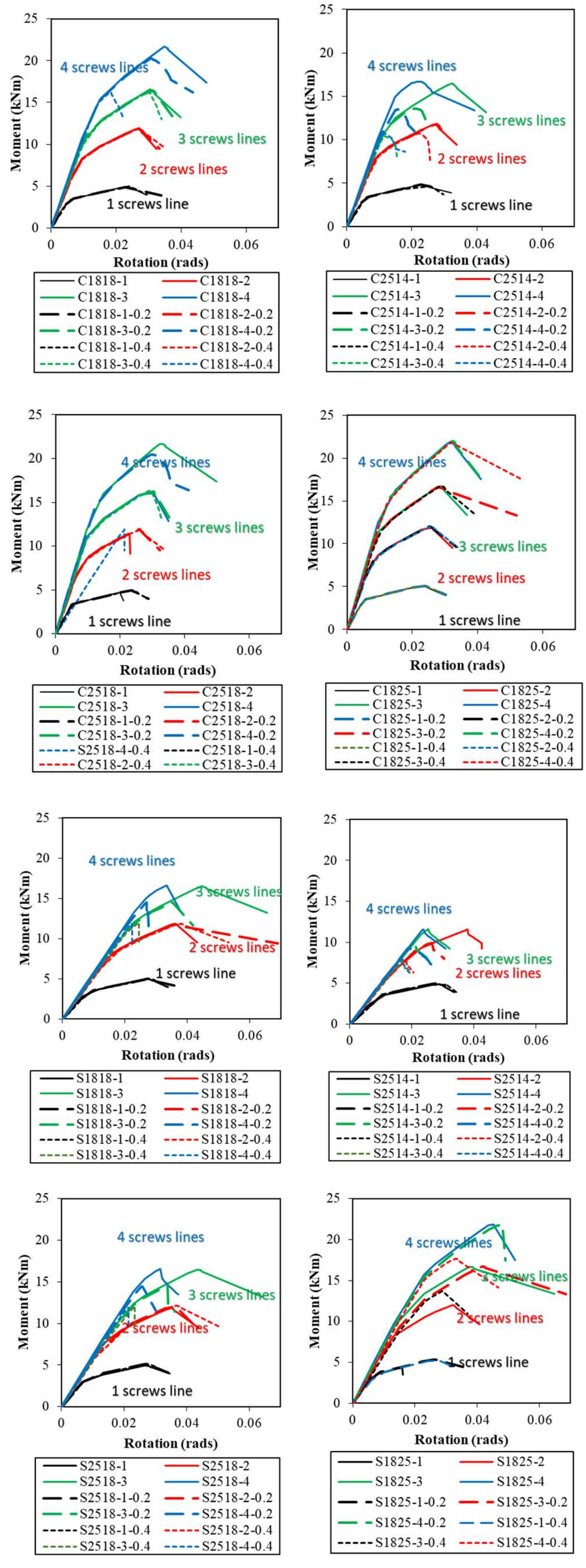


Figure 4. Moment rotation curves for all connection assemblies

4.1 Effect of Gravity Loading

The impact of gravity loading from upper floors on studs was analysed to assess the connection's sensitivity to it. The application of gravity loading increased the internal force in studs, making them more critical. On average, a moment capacity reduction of 11% was observed by adding gravity loads to connection assemblies. The average stiffness of the CCM and SCM connections were 1000 kNm/rad and 530 kNm/rad, respectively in the absence of gravity loading. It is worth noting that this stiffness reflects the combined stiffness of two joist-to-stud connections within each FE model, with the stiffness of a single connection being half of this value. With the application of 20% and 40% gravity loading, the initial stiffness experienced deteriorations of, on average, 30% and 25% for CCM, respectively and 25% and 27% for SCM, respectively. The initial stiffness of the connection assemblies are compared in Figure 5.

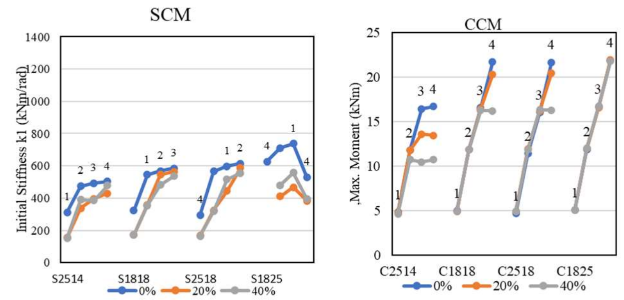


Figure 5 Effect of gravity load on initial stiffness of the connection

4.2 Effect of Screw Arrangements

Four different screw arrangements were implemented adopted in both CCM and SCM connection assemblies. The effects of these arrangements on the initial stiffness and bending moment capacity of the connections with 0% gravity load are illustrated in Figure 6, respectively. In general, the results demonstrated that introducing additional lines of screws can significantly enhance the initial stiffness and maximum bending moment capacity of the connections. There is, on average, a 22% increase in connection stiffness and a 65% increase in maximum moment capacity, respectively, with an increasing number of screw lines.

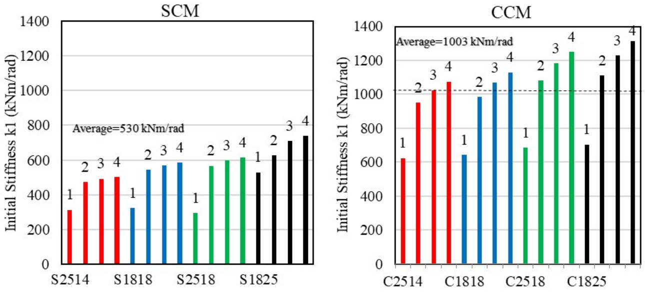


Figure 6. Effect of screw lines on initial stiffness of connection

4.3 Effect of Joist and Stud Thicknesses

No distinct trend was observed for different combinations of thicknesses for studs and joists, as shown in previous figures. However, specimens with joist thickness less than or equal to stud thickness performed better in terms of moment resistance. This performance is attributed to the shift of the governing failure mechanism away from studs towards screws and possibly the joist in such connections. The stiffness of the connection remained mostly unaffected by variations in the combination of joist and stud thicknesses, as the number of screws primarily governs the initial stiffness. This is because the stiffness of single screwed joint is function of screw diameter and thickness of thinnest steel sheet, which were same for S or C1818,2518 and 1825 connections. For S or C2514, a slightly less stiffness was observed (see Figure 6) for all 4 lines for screw due to thinner steel sheet of 1.4 mm.

4.4 Effect of Construction Method

Transitioning to CCM from SCM did not improve the maximum bending moment capacity for connections with joist-to-stud fastener failure as the governing mechanism. However, in connections governed by stud failure, transitioning to CCM increased the maximum bending moment capacity by 23% on average. This improvement is likely due to the beneficial effect of stud continuity in CCM, offering better restraint at mid-height of studs.

5 Case Study – Building Design Example

To illustrate the advantages of utilizing the proposed joist-to-stud connections, a case study design for a typical six-story residential building was conducted. The building had dimensions of 12 m x 30 m, and each floor considered a dead load of 1 kN/m² and a live load of 2 kN/m². A wind speed of 22 m/s was taken into account, resulting in a maximum wind pressure of 1.02 kN/m². Additionally, an earthquake load with a peak ground acceleration of 0.15 g was assumed. The floors were constructed using lightweight concrete and were supported by joists spaced at 0.6 m. Cold-formed lipped channel sections were employed for both joists and studs, with a maximum clear span of 6 m for the joists and 3 m height for the studs at each story. Strap bracing was added to resist horizontal actions.

5.1 Structural Analysis

A 3D model of the building was created using the SAP2000 software package [10] to analyse the internal forces in the structural elements. The figure below illustrates typical bays of the building in the shorter direction. Full lateral bracing was applied to the top flange of the joist, while mid-height lateral bracing was provided to both flanges of the studs, aligning with common practices in the design of Light Steel Frame (LSF) buildings. A rigid diaphragm constraint was applied to the joists. Strap bracing was modelled as a truss element, considering only the tension strap, as the compression strap would not resist lateral loads due to its slender nature. Lateral loads were only applied in the shorter direction, primarily to account for lateral displacement in the design of the gravity frame (unbraced bays) caused by wind and earthquake loadings.

The initial analysis assumed that the joists were pin connected to the studs, implying no moment transfer from joists to studs. Joists were designed against bending action due to the gravity load on the floor, while studs were designed solely against axial compression action. Under this assumption, a maximum bending moment of 11.74 kNm was obtained at the mid-spans of the joists.

The novel joist-to-stud connection introduces rotational stiffness to resist bending moments, estimated at 228 kN/m for sequential construction and 456 kN/m for continuous construction, based on average values obtained from FE modelling in ABAQUS (as explained in earlier sections). To assess the impact of this connection in a building model, rotational springs were incorporated at the joist ends, assigned initial stiffness derived from FE modelling. From elastic analysis, a 10% reduction in maximum mid-span moments for sequential connections and a 20% reduction for continuous semi-rigid connections compared to simple connections was observed. These reduced moments are then redistributed to the joist ends and ultimately transferred to the stud.

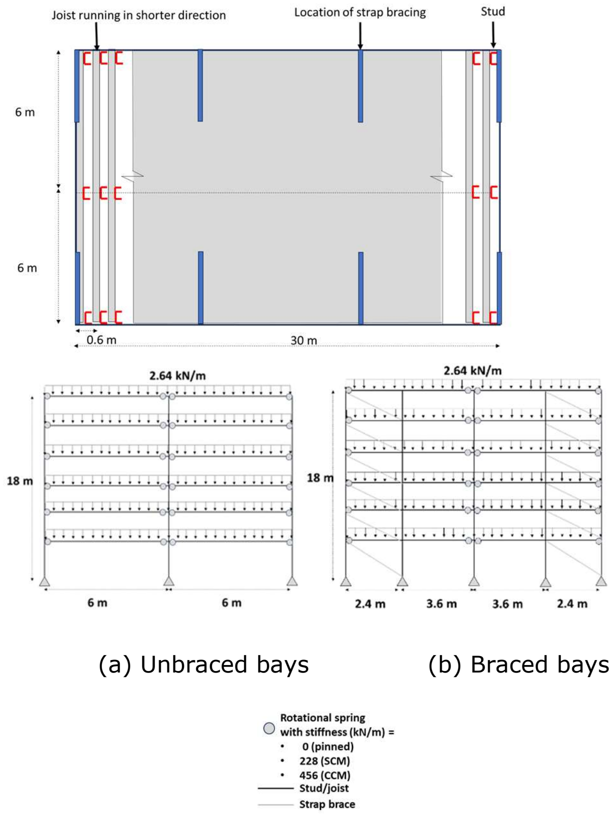


Figure 7. Case stud building models

5.2 Structural Design

Considering internal forces from the elastic structural analysis, joists and studs were designed using Eurocode 3-Part 1-3: EN 1993-1-3 [9]. For semi-rigid connections, studs were also designed against end moments from joists. The structural design summary for three building models is presented in Figure 8. The joist and stud steel material have a yield stress (f_y) of 355 N/mm^2 and ultimate stress (f_u) of 510 N/mm^2. The structural design of the joist is always governed by deflection due to service loads. In the case of semi rigid connections, the capacity of studs was calculated considering moment and axial force interaction.

The use of semi-rigid connections allows for lighter joist sections, reducing mid-span deflections and enabling the transfer of moments from mid-span to supports. Models with sequential and continuous semi-rigid connections require lighter joist sections, resulting in a 22% reduction in steel weight for sequential connections and 28% steel savings for continuous connections compared to models with pinned connections. These outcomes underscore the material-saving benefits of the proposed semi-rigid connection, presenting valuable considerations for the construction industry.

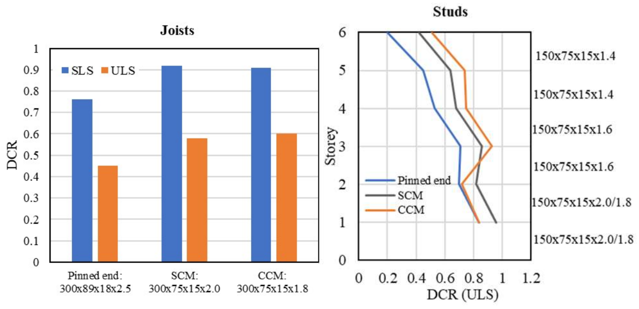


Figure 7. Design summary of case buildings

6 Summary and Conclusions

This paper explores the behaviour of an innovative semi-rigid connection between a joist and a stud, achieved through screwing the web of the joist to the web of the stud. This design obviates the need for ledger tracks and clip angles and is applicable in both continuous and sequential construction methods. The semi-rigidity of the connection enables the utilization of lighter joist sections by mitigating mid-span deflections, a factor typically governing the design of joist elements. The connection's response was analysed through Finite Element (FE) modelling, with validation conducted against experimental results from ledger framed connections in the literature.

A parametric study was undertaken on the FE models to comprehend the impact of key parameters on the response of the semi-rigid connection. The parameters investigated include the presence of gravity loading from upper levels, screw arrangement on the connection, combinations of stud and joist thicknesses, and the type of construction method. The parametric analysis results indicate an average 11% reduction in connection moment resistance with the addition of gravity loads from upper story levels. This reduction is particularly noticeable when the stud is the weaker element of the connection, whereas it is less evident when the joist or screwed connection is the weaker element.

To exemplify the advantages of employing semi-rigid joist-to-stud connections, a case study design for a typical six-story residential building was conducted. The building model assumes either simply supported or semi-rigid joist-to-stud connections, with the latter assigned initial stiffness obtained from FE modelling. Implementing a sequential semi-rigid connection led to a 22% reduction in the steel weight of the structure. Furthermore, the model with a continuous semi-rigid connection demonstrates a further 28% increase in steel savings. These findings underscore the material-saving benefits of adopting the suggested semi-rigid connection, offering significant advantages to the building sector.

References

[1] SCI (2015) *Light Steel Framing in Residential Construction*. SCI Publication P402.

[2] CEN (2005) EN 1993-1-1: Eurocode 3 – *Design of steel structures – Part 1-1: General rules and rules for buildings*. European Committee for Standardization, Brussels.

[3] Ayhan, D.; Schafer, B.W. (2019) *Cold-formed steel ledger-framed construction floor-to-wall connection behavior and strength*. Journal of Constructional Steel Research. 156, pp. 215–226.

[4] Bagheri Sabbagh, A.; Torabian, S. (2021) *Semi-rigid floor-to-wall connections using side-framed lightweight steel structures: Concept development*. Thin-Walled Structures. 160, 107345.

[5] HKS (1996) ABAQUS/Standard User's Manual.

[6] Schafer, B.W.; Ayhan, D.; Leng, J.; Liu, P.; Padilla-Llano, D.; Peterman, K.D.; et al. (2016) *Seismic response and engineering of cold-formed steel framed buildings*. Structures. 8, pp. 197–212.

[7] Shahini, M.; Sabbagh, A.B.; Davidson, P.; Mirghaderi, R. (2019) *Development of cold-formed steel moment-resisting connections with bolting friction-slip mechanism for seismic applications*. Thin-Walled Structures. 141, pp. 217–231.

[8] Tao, F.; Chatterjee, A.; Moen, C.D. (2016) *Monotonic and Cyclic Response of Single Shear Cold-Formed Steel-to-Steel and Sheathing-to-Steel Connections*. Report No. CE/VPI-ST-16-01.

[9] CEN (2006) EN 1993-1-3: Eurocode 3 – *Design of steel structures – Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting*. European Committee for Standardization, Brussels.

[10] CSI (2016) SAP2000 – Structural Analysis Program.