Shear Connection Performance of Cold-formed Steel and Plywood Composite Flooring Systems: Experimental and Numerical Investigation

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Abstract:

This study carries out numerical and experimental investigation on shear connection performance of flooring systems comprising of cold-formed steel joists and engineered plywood panels. A 3-D finite element model has been developed using ANSYS software which considers the nonlinear behaviour and damage of plywood panels using a maximum stress-based criterion incorporated in the FE model. The yielding behaviour of steel joists and fasteners (screws and bolts) is defined as a multilinear plastic hardening material model defined in ANSYS software. The FE model was validated against the experimental results obtained from push-out tests conducted on specimens utilising different connection arrangements. The validated FE models showed a good agreement with the experimental results of the push-out tests in terms of load-slip behaviour, ultimate peak capacity, and failure modes. Finally, the validated FE models are utilised to perform a parametric study that investigates the effect of varying various parameters, including diameter and yield strength of self-drilling screws, the thickness of plywood panels, value of friction coefficient, and the ratio of fastener diameter to the joist thickness, on the initial stiffness, failure mode and ultimate load capacity of the cold-formed steel and plywood composite connections.

Keywords : Composite flooring systems, Cold-formed steel, Shear connection performance, Load carrying capacity, Finite element analysis, Composite beams, ANSYS, Steel, Timber

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1. Introduction

Environmental sustainability is increasingly becoming a key priority in modern construction practices [1, 2]. In addition to minimising the use of materials and resources in structures, engineers focus on limiting the energy consumption and carbon dioxide footprint throughout the life cycle of buildings. As a result, renewable construction materials such as engineered wood products, timber and bamboo are considered in various applications, e.g. composite flooring systems [3].

Timber is an ideal sustainable construction material that poses numerous advantages, including low embodied energy, high strength-to-weight ratio, cost-effectiveness, and environmentally friendly [4-6]. However, the use of timber in long-span structures is limited by its low bearing strength and tension capacity in particular [7]. Furthermore, the fibre orientation of timber panels affects the load transfer in timber connections and their bearing capacity [8-12]. Therefore, researchers have suggested composite structures to overcome these issues [13]. For example, combining timber with other construction materials such as cold-formed steel members improve the timber bearing capacity.

Cold-formed steel members are characterised by low self-weight, high bearing capacity, high ductility, and ease of construction; yet, cold-formed steel members are susceptible to local instabilities due to their thin-walled parts [14-16]. Therefore, the current research project combines cold-formed steel and timber panels to develop a new composite section. The proposed composite section is fabricated by connecting a cold-formed steel C-section with plywood panels (engineered timber) using different shear connectors. The new composite section utilises the strength and ductility provided by the cold-formed steel to complete the bulky and lightweight plywood panels. Additionally, cold-formed steel acts as the primary load-bearing member, whilst the plywood panels provide lateral stiffness for the cold-formed steel and enhance the overall bearing capacity of the section. As a result, the new proposed composite section considerably minimises the use of material and resources in the construction process, reducing the energy consumption and the environmental impact of buildings [17, 18].

Significant research has been carried out to study a variety of composite systems, including the combination of timber with steel, timber with concrete, and steel with concrete. Loss and Davison [19] conducted bending tests to study the performance of composite flooring systems comprised of cold-formed steel sections and cross-laminated timber (CLT). Two types of floors were tested. The first floor used self-drilling screws to connect cold-formed steel sections and CLT panels, whereas the second floor employed steel plates and structural epoxy resin. The study reported that the steel-CLT composite floors demonstrate significant potential considering bearing capacity, stiffness, and construction method. Yang, Li [8] investigated a new hybrid floor system that utilises steel H-section and glulam (engineered timber). Yield properties and failure modes were investigated through a series of push-out tests. Two types of shear connectors were used in the study; self-drilling screws and bolts. They compared bearing capacity test results against the values predicted by several codes, such as Eurocode 5 [20]. The comparison proved that a gap exists in the current knowledge of steel timber connections since the predictions from the codes deviated significantly from the experimental results. Hassanieh, Valipour and Bradford [21-23] conducted a series of push-out tests and four-point bending tests to study the load-slip behaviour and failure modes of steel-CLT and steel-laminated veneer lumber (LVL) composite systems. The study employed different shear connectors, including bolts, screws, steel plates, and structural adhesives to connect steel sections with timber panels. They also carried out a comprehensive finite element analysis to investigate the effects of various parameters on the steel-timber connection behaviour. Hosseinpour et al. [24] investigated the shear connection properties of composite beams comprised of cold-formed steel joists and concrete slabs combined with bolts. Push-out test results showed that ductility, stiffness, and bearing capacity of the tested beams are significantly influenced by the joist's thickness and bolt's diameter and strength. Conversely, a Pull-test procedure to determine the shear behaviour and strength of a connection between timber studs and plasterboards is proposed by EN 520 [25]. Few studies [26-29] adopted the EN 520 procedure to measure the connection behaviour of CFS profiles sheathed with gypsum boards, cement boards, or plasterboards. The thickness of boards used in these studies varied between 11mm and 18mm.

Similar results were obtained by [30, 31]. Du et al. [32] studied the structural performance of glulam-concrete composite floors under static shear tests. The study demonstrated that inclined crossing screws are an attractive fastener layout to combine the structural components of the glulam-concrete composite floors. They also illustrated that using a larger screw diameter with a longer embedment length into the glulam joist improved the bearing capacity of the connection. Similar results were also observed by [33-35]. Recently, Kyprianou et al. [36, 37] have conducted a comprehensive experimental and analytical study on the connection mechanical behaviour of sheathed cold-formed steel studs. Different sheathing products were investigated, including Oriented Strand Boards (OSB) and particleboards. Analytical models were proposed based on two stage Ramberg-Osgood model [38-40] considering the geometries of the fasteners and sheathing material and the strength of boards.

Selvaraj and Madhavan [41] carried out comprehensive research to study the torsional buckling behaviour of CFS joists sheathed with various types of boards. Forty-two wall panels were tested utilising seven different types of sheathing boards, six different CFS geometries, and various types of connectors. It was concluded that fibre-composed sheathing boards performed better to prevent torsional buckling of CFS joists compared to the performance of particleboards. Additionally, the geometry of the CFS influenced the behaviour of the sheathing material. The study also suggested a sheathing material that provides higher resistance against the torsional buckling failure mode of CFS joists. Analytical formulation that predicts the stiffness of the sheathing boards as a function of its tensile modulus and the dimensions of the CFS joists was proposed by [42, 43].

Indeed, former research studies primarily used engineered timber products such as CLT, LVL and Glulam that are produced from European softwood species or, occasionally, from Chinese grown wood species; however, limited research utilising plywood panels have been conducted. Specifically, research studies of composite floors comprised of plywood panels and cold-formed steel joists are scarce. The work presented in this paper demonstrates the connection behaviour of composite floors fabricated using Structural plywood and cold-formed steel joists, extending the existing research of composite floors accordingly. The structural plywood panels and cold-formed steel joists used in the current study are manufactured from local Australian resources. The plywood panels and cold-formed steel joists are produced with different thicknesses and structural purposes and are widely used in the Australian local market owing to its availability and affordable price tag. Additionally, self-drilling screws (SDS), coach screws, and bolts were used to fabricate the composite section. Structural adhesive is also employed in conjunction with mechanical fasteners to connect plywood panels with cold-formed steel joists.

Attained degree of composite action is primarily determined by the mechanical performance of shear connectors [44-46]. In addition, the design of structural connections influences the durability and stability of the structure [8]. In composite construction, dowels are mainly designed to resist horizontal shear forces between the joist flange and top slab and endure the uplift loads between them. Accordingly, the current study investigates the failure modes and the mechanical properties of the composite cold-formed steel and plywood connections by conducting a series of push-out tests. Push-out tests were carried out in double-shear and symmetrical configurations where two back-to-back cold-formed steel C-sections are joined to two plywood panels. In addition to examining the bearing and yield capacities of the proposed composite section, the stiffness, failure modes, ductility, and effects of using different fasteners and spacing are also analysed. Besides, a finite element model (FEM) is developed and validated against the experimental results. The FEM is then extended to explore the effects of variations in several parameters, including yield strength of fasteners, the diameter of SDS, coefficient of friction between plywood and joist's flange, and thickness of plywood panels. Finally, results obtained from this research will be used in future works to develop analytical formulae, recommend practical design guidance, and promote future research works on cold-formed steel-plywood composite sections.

2. Experimental program

2.1 Materials

The plywood panels used in this study were manufactured from a locally sourced plantation Radiata pine following AS/NZS 2269 [47] with panel dimensions of 2400 mm × 1200 mm × 45 mm and a moisture content of 10–15% after drying. Besides, phenolic adhesive was used to produce a type-A bond between the plywood strata following AS/NZS 2754.1 [48] and AS/NZS 2098.2 [49]. Bending, compression, and tension tests were carried out on the plywood panels according to AS/NZS 2269.1 [50], and the basic mechanical properties are provided in Table 1. At least four identical tests were performed for each test type, and the loading rate was adjusted such that specimens failed within 3 to 5 min.

Joists used in the current research were a grade G450, cold-formed steel C-section manufactured to meet provisions of AS/NZS 4600 [51]. The joists were rolled-formed from a galvanised steel sheet with a thickness of 2.4 mm and a minimum zinc coating mass of 350 g/m^2, which meets AS 1397 [52] requirements. The dimensions of the C-sections are provided in Fig.1, where l denotes the length of the joist, l = 500 mm. Moreover, to determine the mechanical properties of steel joists, twelve flat coupons were extracted from the cold-formed steel sections and tested following AS 1391 [53]. Fig. 2 shows the tensile steel coupons' dimensions and locations, whereas the cold-steel joists' basic mechanical properties are presented in Table 2.

Lastly, three types of fasteners were used in the testing program, including bolts (diameter: 8mm and 12 mm, length: 75 mm, grade: 4.6), coach screws (CS) (diameter: 12 mm, length: 45 mm, material: low carbon steel), and Self-drilling Screws (SDS) (diameter: 6 mm, length: 47 mm, material: low carbon steel). All fasteners were procured from the local market and were manufactured according to relevant Australian standards [54, 55]. The dimensions, proof yield strength, and ultimate tensile strength of the shear connectors are provided in Table 3.

2.2 Geometry and details of test specimens

As part of the current research, 24 specimens were fabricated and tested. The specimens were categorised into eight groups, and each group had three identical specimens. Groups A and B had identical specimens, but connector type was varied as 6 mm SDS and 12 mm coach screws. Specimens in groups C and D varied only in bolt diameter, i.e. 8 mm and 12 mm, whilst connector type, grade, and connector spacing were maintained identical. Group E specimens are identical to group D specimens, except washers were not utilised to fabricate group E specimens. Specimens of the remaining three groups (F, G, and H) employed structural epoxy resin in conjunction with 6 mm SDS, 12 mm coach screws, and M12 bolts, respectively. In the fabrication of the samples, pilot holes were pre-drilled into the cold-formed steel joists and plywood panels. For bolted and coach screw connections, the diameter of the pre-drilled holes in the plywood panels was 1 – 2 mm larger than the bolt's diameter and 2 mm smaller than the coach screw diameter. Pre-drilled holes in the cold-formed steel joists had a diameter of 0.5 mm greater than the diameter of the connectors (screws and bolts). Particular considerations were taken to ensure the holes in plywood and joists were perfectly aligned. Pre-drilling of plywood was unnecessary for specimens with SDS dowels as screws were directly drilled into the panels. Finally, a post-tension force equal to $0.1f_{yb}$ was induced in bolt shear connectors. Washers were used in conjunction with bolted connections to prevent plywood crushing when post-tensioning force is applied to the bolts. Washers were 50×50×4 mm with 240 MPa yield strength and 400 MPa ultimate strength.

Table 4 provides the basic parameters of specimens in each group, whilst Fig. 3 shows the connection arrangements of tested specimens. In all specimens, dowels were arranged symmetrically to prevent undesirable effects during testing, such as unequal load distribution. The arrangement was divided into two rows, and each row had two dowels with a total of 8 connectors for each sample. For all specimens, spacing of connectors, edge distance, and end distance comply with provisions specified in Australian standards [51, 56].

2.3 Test setup and loading scheme

Four linear variable differential transformers (LVDTs), with a stroke of ±50 mm, were used to measure the relative slip between the joists and plywood. The LVDTs were positioned at the interface between the cold-formed steel sections and plywood to minimise the influence of the unequal distribution of load and material on the slip measurement. The body of the LVDTs was fixed on the plywood panels, whilst their measurement needles were attached to the joists through steel angle. Configurations of LVDTs are shown in Fig. 4.

Side edges of plywood panels were accurately cut to ensure they maintain uniform contact with the smoothed surface of the testing platform, reducing the effects of imbalanced stress on the slip measurement. Furthermore, a 50 mm thick steel plate was used to transfer the applied load equally through the entire cross-section of the joists. The use of the steel plate helped prevent the occurrence of local buckling in the web. A typical test setup is shown in Fig.5. Load-slip relationship and modes of failure were the key results obtained from the push-out tests. Tests were conducted using a microcomputer-controlled electro-hydraulic servo universal testing machine with a capacity of 100kN and a TDS-530 data acquisition system.

The adopted loading scheme is based on the recommendations of BS EN 26891 [57]. The load was applied in two stages. The testing was carried out under a load-control procedure in the first stage until 70% of the estimated ultimate load (P_{est}) was achieved. Beyond $0.7P_{est}$, a displacement-control procedure was applied, and the testing continued until the specimen failed. A standard loading scheme is shown in Fig. 6. Initially, the load was applied up to $0.4P_{est}$ at a constant rate of $0.2P_{est}$ and then maintained for 30 sec. After that, specimens were unloaded, the load was decreased to $0.1P_{est}$, and the load was kept constant for 30 sec. Finally, the load was increased until specimen failure was reached. According to BS EN 26891, failure of the specimen is depicted by reaching the ultimate load (described by crushing of timber, yielding of fasteners, or both) or a slip of 15 mm.

3. Discussion of experimental results

3.1 Failure modes

Generally, push out specimens comprised of different materials and different types of connectors exhibit four distinctive mode of failures [8, 57-59], as illustrated in Fig. 7. Failure mode I is mainly characterised by timber crushing in the vicinity of the fasteners while fasteners experience minor deformation. Failure mode II is associated with forming a plastic hinge within the dowels and crushing timber elements in contact with the connectors. Failure mode III is similar to the second failure mode except that two plastic hinges form in the connectors. Failure mode IV is described by a shear failure of the dowels accompanied by considerable timber crushing. Finally, two factors determine the likelihood of each mode failure; dowels slenderness given by *L/D* (*L* is the dowel length, and *D* is its diameter), and strength ratio f_s/b_s (f_s is the shear strength of the connector, and b_s is the bearing strength of the primary element) [22]. Failure modes II and III are considered ductile, while failure modes I and IV are deemed brittle modes of failure.

Fig. 8 shows typical failure modes observed in the tested specimens. In all tested specimens, crushing of timber was observed in the locality of the connectors (Fig. 8a). As the load was increased, connectors (screws or bolts) started to squeeze the timber in contact with them once the applied load surpassed the friction between the joists and plywood, resulting in local damage to the plywood panel and deformation of the dowels. Push out specimens utilising SDS (Group A) connectors were dominated by mode failure IV, where the screws experienced shear failure associated with timber crushing (Fig. 8a). The failure mode in specimens of groups B and E was characterised by forming a plastic hinge in the dowels, and a local bearing failure of timber in the direction perpendicular to the grain, indicating that failure mode II was dominant in these groups (Figs. 8b and 8d). Similarly, the formation of two plastic hinges and localised timber crushing under the connector head was observed in groups C and D specimens (Figs. 8c and 8d).

Furthermore, M8 bolts exhibited shear strength failure at the shear plane between plywood and joists (Fig. 8c). Specimens in groups F, G, and H showed a brittle failure associated with de-bonding of the structural adhesive once the load reached 60% of the ultimate load.

In general, specimens utilising M12 bolts or screws exhibited ductile failure, whereas specimens with SDS dowels suffered a brittle failure. Accordingly, observations of failure modes in specimens that used M8 bolts as connectors indicated that these specimens exhibited ductile and brittle failure. It is highlighted that the joists and connector holes did not experience any deformation. However, a slight kink around pre-drilled holes was observed in specimens using M12 screws or bolts.

3.2 Load-slip curves of the connections

Load-slip curves for specimens of different groups are plotted in Fig. 9. Generally, four stages can be defined to describe the load-slip curves of the tested specimens, as illustrated in Fig. 10.

The first part of the load-slip curve is the no-slip part. At this stage, the composite push out specimens behaved as a fully composite system with relatively negligible or no slip occurring at the interface due to the friction between plywood and joists. In specimens with bolted joints, friction was achieved by post-tensioning the bolts. In contrast, the threads of the SDS connectors firmly locked the joist flange to the plywood panel, which produced substantial post-tensioning force. It is highlighted that the post-tensioning of bolts was calibrated against the compressive strength of the plywood to prevent local crushing of timber in the direction perpendicular to the grain.

The initial stiffness of the push-out specimens is defined by the load at which the first slip occurs. The force required to initiate slip at the interface depends on the magnitude of the friction force that exists between plywood panels and joists. Specimens with SDS joints began to slip at an average load of 3.11kN, whilst the observed load for specimens utilising CS was 8.14kN. Similarly, the average loads that initiated slip in specimens of groups C, D, and E were 4.72kN, 18.2kN, and 9.81kN, respectively. The additional initial resistance of M12 bolted joints with washers (Group D) is attributed to the applied post-tensioning force, which considerably enhanced the friction at the interface of the composite section. The initial stiffness of specimens utilising structural epoxy resin was determined based on the inherent properties of the epoxy and the quality of workmanship. For SDS connected joints with epoxy, the initial slip was observed at an average load of 6.10kN, whilst for CS and M12 composite joints, the average load was 10.55kN.

The second stage of the load-slip curve was the elastic stage, where the applied shear force exceeded the friction at the interface, and the slip increased relatively proportional to the increased load. Once the applied load nearly attained 45% of the peak load, excessive slips occurred at a lesser increase in the load, triggering the nonlinear behaviour of the composite system and eventually reaching the peak load. Finally, when the maximum load was attained, the load-slip curve started to descend, initiating the failure of the specimen.

3.3 Bearing capacity of push-out specimens

The key test results of the push-out specimens are calculated according to BS EN 12512 [60] guidelines. As illustrated in Fig. 11, the ultimate load F_u is taken as 80% of the peak load $F_{max.}$ The slip value corresponding to the intersection of a horizontal line drawn from the F_u is considered the ultimate slip V_u of the specimen. Furthermore, the vertical and horizontal coordinates of the intersection point between secant line I and secant line II are considered as yield load and yield slip, respectively. Moreover, the service state slip stiffness $K_{0.4}$, the ultimate bearing state slip stiffness $K_{0.6}$, and the failure state slip stiffness $K_{0.8}$ are calculated to reflect on the performance of the shear connectors. Finally, the working ductility of the shear connectors D is determined as the ratio of ultimate slip to the yield slip. Key results of the tested specimens of different groups are tabulated in Table 5.

4. Numerical model and parametric study

4.1 Materials

4.1.1 Plywood

Plywood panels are 45 mm thick and consist of 3 mm thick spruce wood layers, compressed and glued together. However, owing to the small thickness of plywood laminates, the laminate-wise nature of the plywood was not considered, and the Plywood panels were modelled as a homogenous material.

Plywood panels were treated as an orthotropic material whilst the nonlinear behaviour and failure of the panels under multiaxial stress state were captured using the Maximum Stress Damage Initiation Criteria available in the ANSYS library of material. The maximum stress criteria are based on the framework of continuum damage mechanics developed by Sandhaas [61]. The formulated constitutive law can simultaneously simulate the brittle and ductile failure of wood material by identifying several failure modes, including tensile, compressive, and shear failure. The stress strain relationship of the Plywood panels are given in Fig. 12. The constitutive law treats plywood as an orthotropic material. However, the mechanical properties of the plywood are assumed to be identical in all directions, i.e. longitudinal, perpendicular, and tangential directions. The elastic properties of the plywood panels (Young's modulus, shear modulus, and Poisson's ratio) and strength properties (Bending strength and compression strength) are provided in Table 1.

4.1.2 Cold-formed steel and fasteners

Three steel components are used to develop FE models, i.e., steel joist, shear connectors (bolts and self-drilling screws), and washers used in conjunction with bolted connections. Steel joists used to fabricate push out specimens were cold-formed steel C-sections with a nominal thickness of 2.40 mm. Yield strength, ultimate strength, and elastic modulus of the joists are provided in Table 2. Accordingly, bolts and self-drilling screws utilised in the validation study were made of 4.6-grade steel with a yield strength of 250 MPa and ultimate strength of 400 MPa. Table 3 provides the material properties of shear connectors adopted in the numerical study. For all modelled steel components, Poisson's ratio was taken equal to 0.3, whereas the elastic modulus was assumed equal to 205 GPa. Bilinear isotropic hardening [39] was adopted to model the nonlinear behaviour of the cold-formed steel joist, whilst multilinear isotropic hardening [62] was used to simulate the plasticity of shear connectors. The modified two-stage Ramberg-Osgood constitutive model proposed by Gardner and Ashraf [39], presented in Eqs. (1) and (2), was adopted in the FE model to describe the material behaviour of the cold-formed steel joists.

$$\varepsilon = \frac{\sigma}{E} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^{n} \text{ for } \sigma \leq \sigma_{0.2} \qquad \text{Eq. 1}$$

$$\varepsilon = \frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \left(\epsilon_{1.0} - \epsilon_{0.2} - \frac{\sigma_{1.0} - \sigma_{0.2}}{E_{0.2}}\right)\left(\frac{\sigma - \sigma_{0.2}}{\sigma_{1.0} - \sigma_{0.2}}\right)^{n'_{0.2,1.0}} + \varepsilon_{0.2} \text{ for } \sigma_{0.2} < \sigma \leq \sigma_{u} \qquad \text{Eq. 2}$$

Where $\epsilon, \sigma, and\ E$ are the engineering stress, strain, and Young's modulus of the material respectively, $\sigma_{0.2}$ and $\sigma_{1.0}$ are the proof stresses at 0.2% and 1% strain, $\epsilon_{0.2}$ and $\varepsilon_{1.0}$ are strains of the corresponding proof stresses, and n and $n'_{0.2,1.0}$ are the parameters describing the roundedness of the stress-strain curve. The stress-strain curves implemented in the FE simulation for joists and fasteners are provided in Figs.13 and 14. Finally, a post-tension force of $0.1f_y$ was induced in bolted connections to replicate the actual testing conditions, and 50×50×4 mm washers were modelled to prevent the crushing of timber panels.

4.2 Geometry and boundary conditions

The arrangements of tested push-out specimens (Fig. 3) utilise symmetry to prevent unfavourable behaviour during testing, particularly unequal load distribution. Accordingly, one-half of the tested specimens is adopted in the FE model to reduce the computational time, with appropriate boundary conditions applied at the symmetry plane. Moreover, the movement of plywood panel sides (Fig.15) was restrained in the x, y, and z directions. The displacement of the joist's web is restrained in the x and y directions, whilst a z-direction displacement is applied to the flanges of the cold-formed steel joist. Boundary conditions described in this section and implemented in the FE simulation are shown in Fig. 15.

4.3 Modelling of contacts

The contact behaviour between the different components of the push-out specimens, including plywood panels, cold-formed steel joists, and fasteners, is modelled using surface-to-surface frictional contact. For all contacts, the interface treatment contact option was set to "Adjust to Touch" to ensure contact surfaces can slide, do not interpenetrate through each other, and transmit normal compressive forces and tangential forces. Tangential forces were transferred between contact elements using a penalty-based contact formulation in which the Coulomb coefficient of friction is assigned to the surfaces in contact. Three different values for the coefficient of friction μ were adopted in the FE model, i.e. μ is equal to 0.25 for plywood-to-joist contact, 0.25 for plywood-to-fasteners contact, and 0.35 for steel-to-steel contact [63, 64].

4.4 Meshing and element types

The choice of suitable mesh size can significantly affect the efficiency and precision of the developed numerical models [61]. Although a fine mesh size accurately represents the physical tests and reduces the formation of hourglass modes, the FE model may be computationally expensive, rendering the model inefficient [44]. Accordingly, mesh sensitivity analysis is carried out to define an appropriate mesh size. Three different sizes are analysed, i.e. fine, medium, and coarse. The ultimate load and initial stiffness predicted by the FE model are compared against the test results of group A specimens. It is apparent from Fig. 16 that the mesh size slightly influences the ultimate peak load and initial stiffness. Therefore, the medium mesh size density is selected since it provides a reasonable compromise between the precision and efficacy of the validated FE model. An outline of the adopted FE mesh is shown in Fig. 17.

Shell elements (SHELL181) are used to mesh cold-formed steel joists [62]. SHELL181 is suitable for analysing thin to moderately-thick shell structures and can capture high plastic deformations. A higher-order 3-D 20-node solid element that exhibits quadratic displacement behaviour (SOLID186) is adopted to mesh fasteners and plywood parts in contact with fasteners. The SOLID186 [62] element supports the plasticity and large deflection capabilities and is well suited for irregular meshes such as fasteners and plywood regions with holes incorporated in the developed FE model. The remaining parts of the plywood panels are meshed using a lower order solid element, i.e. an 8-node linear solid element (SOLID185) [62] which is well suited for general 3-D solid structures. Fig.18 shows the solid structural geometry of SOLID185 and SOLID186 elements [62].

4.5 Analysis scheme

The applied load on the FE models was carried out using a displacement-controlled scheme, with the displacement applied at the edges of the cold-formed steel joist. The displacement-controlled procedure allowed the FE models to reach ultimate load capacity and trace the post-buckling behaviour of the systems. The displacement was conducted following step by step procedure. Furthermore, the general static mechanical solver was employed to analyse the FE models described in this research study due to the unstable behaviour of cold-formed steel joists and large contact surfaces. In bolted connections, a post-tension force was induced in the bolts, where the force was applied in the second and third (final) steps to reduce the effect of contact initiation on the solution convergence [45]. Finally, in the case of screw fasteners, the contact interaction between plywood and joist was found to cause convergence problems. Hence, the un-symmetric Newton-Raphson option was adopted during the solution.

4.6 Validation of the FE model

The adequacy of the developed FE model in predicting load-slip relationship, peak load capacity, ultimate slip, and deflection at the failure of push-out specimens has been verified against the available experimental results.

The load-slip curves obtained from the FE model for push-out specimens connected with self-drilling screws were compared to the experimental load-slip responses as shown in Fig. 19. Moreover, the peak load capacity predicted by the FE model is also compared with the experimental findings (Fig. 19). Predictions from the FE model show a good agreement with the experimental results. In particular, the FE models captured the post-buckling behaviour of the load slip curve. However, the FE model overestimates the peak capacity of the push-out specimens. The overestimation of the peak capacity is attributed to the assumed elastic-perfectly plastic behaviour of the plywood in compression, which poorly represents the softening behaviour of the timber. Timber elements in contact with fasteners exhibit higher stiffness than the physical tests resulting in a larger peak load capacity predicted by the FE model.

The permanent deflection of 6 mm SDS composite connections predicted by the FE model is compared with the deformations observed in the tests, as illustrated in Fig 20. Accordingly, the permanent deformations captured by the FE model correlate reasonably well with physical tests.

5. Parametric studies and discussion

The validated FE model is utilised to investigate the influence of diameter and yield strength of self-drilling screws, the thickness of plywood panels, the value of friction coefficient, and the ratio of fastener diameter to the joist thickness on the initial stiffness, failure mode and ultimate load capacity of the cold-formed steel and plywood composite connections.

5.1 Effect of fastener diameter

The diameter of the SDS has a significant effect on the load-slip behaviour of the cold-formed steel and plywood composite connections. Accordingly, the influence of different screw diameters (d_s = 6, 8, and 10 mm) on the load-slip response (Fig. 21) was investigated. As demonstrated in Fig. 21, the diameter of the screw changes the initial stiffness and peak load capacity of the connection. In regards the ultimate bearing capacity, FE models with 8 mm and 10 mm diameter SDS showed 80% and 279% increase in the ultimate capacity, respectively, compared to FE models utilising 6 mm diameter screws. Similarly, 8 mm and 10 mm FE models were 20% and 46% stiffer than FE models with 6 mm screws. Larger screw diameter provided higher contact areas between the fastener, plywood panels, and cold-formed steel joists which resulting in improving the rigidity of the connection and ultimately enhancing the overall behaviour of the composite connection. Additionally, higher peak load capacity and stiffness are expected for larger screw diameters due to the larger cross-sectional area of the fasteners.

5.2 Yield strength of self-drilling screws

Fig. 22 shows the load-slip curves of cold-formed steel and plywood composite connection with different yield strengths of SDS, f_y. The investigated values range between 120, 240, and 360 MPa. It is evident that the ultimate strength of the connections is proportional to the yield strength of the self-drilling screws f_y. FE models using 360 MPa fasteners showed a 22% higher ultimate capacity, whilst the models using 120 MPa connectors had a 26% lower bearing capacity than numerical models with 240 MPa fasteners. On the other hand, the initial stiffness of the analysed composite connections with 120 and 360 MPa yield strengths was slightly affected compared to connections utilising 240 MPa yield strength fasteners. Accordingly, similar observations were reported by Hosseinpour, Zeynalian [24]. Aune and Patton-Mallory [65] proposed a simplified formula relating the square root of the yield strength of the fasteners to the ultimate bearing capacity of the connection (Eq.3).

$$F_u = \sqrt{2f_e M_y} = 0.25\sqrt{\pi d^3 f_e \sigma_y} \qquad \text{Eq. 3}$$

However, the effect of fastener size and installation method were not considered in Eq. 3. Therefore, Eq. 3 provides only an approximation of the peak capacity of the connection.

5.3 Thickness of plywood panels

The validated FE models were used to study the influence of the plywood panels thickness on the load-slip behaviour of composite connection with self-drilling screws. Three different plywood thicknesses (45, 32, and 25 mm) available in the Australian market were investigated. The load-slip curves presented in Fig. 23 demonstrate the minor influence of plywood thickness on the initial stiffness of the composite connection. In contrast, the post-buckling behaviour of the composite connections was significantly influenced by the change in the plywood thickness. FE models with 32 mm and 25 mm thick plywood panels showed an 11% and 25% drop in the peak ultimate capacity, respectively. Both models failed prematurely before reaching higher slip values than FE models utilising 45 mm thick panels. Thinner plywood panels have a lower resisting contact area; therefore, higher stress concentrations are accumulated at regions in contact with the fasteners. Consequently, plywood slab fails before fasteners reach yielding.

5.4 Coefficient of friction between joist and plywood panels

The friction between plywood slab and joist flanges can affect the initial slip and ultimate load-bearing capacity of composite connections utilising dowel-type connectors. Accordingly, the load-slip behaviour of push-out specimens utilising SDS fasteners with various coefficient of friction values, i.e., $\mu = 0.2$, 0.3, and 0.4, were studied using the FE models as shown in Fig. 24. It is observed from Fig. 24 that the coefficient of friction between plywood panels and joists had an insignificant effect on the initial stiffness of the connections. However, it had a slightly higher impact on the ultimate capacity of the connection. Composite connections with a higher coefficient of friction experienced a marginally higher peak capacity values.

5.5 Influence of the SDS diameter to joist thickness

The load-slip behaviour of the composite connection and its mode of failure may be affected by the ratio of connector diameter (d_s) to the joist thickness (t_f). Generally, failure of steel-timber composite connections is limited by the yield strength of the fasteners or crushing in timber. However, localised plastic stresses and bearing failure of the joists tend to dominate the failure mode of the connection as thinner joist flanges are used. Accordingly, three different joist thicknesses (1.5, 2.4, and 4.2 mm) were studied using the SDS 6 mm FE model. The load-slip behaviour of d_s/t_f ratios of 4, 2.5, and 1.5 are shown in Fig. 24. It is observed that the d_s/t_f ratio had an insignificant effect on the initial stiffness of the connection. But, the peak bearing capacity of the connection was slightly reduced as the d_s/t_f ratio exceeded 2.5 because of the localised strains formation and bearing failure around screw holes in the cold-formed steel joist.

6. Concluding remarks

In this study, nonlinear finite element models of composite connections comprised of plywood panels and cold-formed steel joists were developed and validated using ANSYS software. The developed FE model considers material nonlinearities of cold-formed steel and fasteners, including yield strength and plastic hardening. It also considers the nonlinear frictional contacts between various components of the FE model, including the contact between connectors and plywood, contact between joists and fasteners, and, particularly, at the interface between the cold-formed steel joists and plywood panels. The developed FE model was validated against available experimental results conducted using push-out specimens utilising various arrangements of shear connectors. It was shown that the developed FE model accurately captures the load-slip behaviour of the push-out specimens in terms of initial stiffness, post-yield behaviour, and ultimate load-bearing capacity.

Additionally, the developed FE model precisely predicts the failure modes of plywood panels and fasteners observed in the physical tests. The validated FE model was then used to investigate the effect of various parameters on the initial stiffness and peak ultimate loads of the composite connections. The following conclusions can be drawn from the parametric study:

- Changing the diameter of the fastener has a significant effect on the initial stiffness and ultimate peak capacity of cold-formed steel and plywood composite connections.
- The yield strength of the fasteners (f_y) considerably influences the peak capacity of the connection. The ultimate bearing capacity was found to be proportional to the square root of fastener yield strength.
- The thickness of plywood slab (length of screw connector) has an insignificant effect on the behaviour of the composite connection in terms of initial stiffness, failure mode, and ultimate bearing capacity.
- Three different friction coefficients were adopted at the interface between the plywood panel and cold-formed steel joist, i.e. 0.2, 0.3, and 0.4. Varying the coefficient of friction has a negligible influence on the initial stiffness of the composite connection but slightly changes the ultimate capacity. Connections with higher coefficients showed higher ultimate load capacity.
- The ratio of fastener diameter to the joist thickness d_s/t_f influenced the failure mode of the connections and peak capacity. FE models with higher *ds/tf* values showed that localised plastic strains and bearing failure around fasteners holes in the steel joists were more dominant than yielding fasteners or crushing timber failures. The ultimate load capacity was marginally decreased when higher values of models with higher values of d_s/t_f ratio were used. However, the initial stiffness was insensitive to the *ds/tf* ratio change.
- Based on the experimental results and parametric study results, the main factors affecting the mechanical behaviour of cold-formed steel and plywood composite connections are the properties of connectors, particularly the yield strength (f_y) and diameter of the fasteners (d_s).

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Table 1. Mechanical and strength properties of plywood panels

Air-dried density (kg/m3)	Bending strength (MPa)	Parallel-to-grain tensile strength (MPa)	Parallel-to-grain compression strength (MPa)	Elastic modulus (MPa)	Modulus of rigidity (MPa)	Poisson's ratio	Moisture content %
500	40	5	30	10,200	40	0.2	11.5

Table 2. Mechanical properties of cold-formed steel joists

Section depth (mm)	Thickness (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Elastic modulus (MPa)	Elongation at fracture ε (%)
254	2.4	500	560	205,000	13

Table 3. Material properties of connectors

Fastener type	Diameter, *d* (mm)	Length, *l* (mm)	Grade	Yield strength (MPa)	Ultimate strength (MPa)	Elastic modulus (MPa)
Self-drilling screws (SDS)	6	45	-	280	450	
Coach screws (CS)	10	40	-	280	450	205,000
Bolts	8	65	4.6	250	400	
	12	75	4.6	250	400	

Table 4. Basic parameters of specimens tested in this study

Group ID	Connection type	Diameter, *d* (mm)	Length, *l* (mm)	Spacing, *s* (mm)	Number of specimens
Group A	SDS	6	400	200	3
Group B	CS	12	400	200	3
Group C	Bolts	8	400	200	3
Group D	Bolts	12	400	200	3
Group E	Bolts - No washers	12	400	200	3
Group F	SDS + adhesive	6	400	200	3
Group G	CS + adhesive	12	400	200	3
Group H	Bolts+ adhesive	12	400	200	3

Table 5. Key test results (mean values) for specimens with different shear connectors

Group no.	Load at failure, P_u (kN)	$K_{0.4}$ (kN/mm)	$K_{0.6}$ (kN/mm)	Ductility, D	Normalised stiffness	
					$K_{0.4}$ (kN/mm)	$K_{0.6}$ (kN/mm)
Group A	32.2	17.59	16.99	4.8	2.19	2.12
Group B	82.8	10.89	10.66	5.3	1.36	1.33
Group C	67.6	20.2	20	6.1	2.52	2.5
Group D	183.5	44.4	36.8	11.2	5.55	4.6
Group E	132	21.96	18.26	7.1	2.75	2.28
Group F	34.6	90.5	73.7	23.4	11.31	9.21
Group G	110	52	29.4	15.7	6.5	3.6
Group H	140.8	55.7	31.5	19	6.96	3.93