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Research on cold-formed steel connections: A state-of-the-art review

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Abstract. Cold-formed steel structures are increasingly attractive due to their benefits of good mechanical performance and constructional advantages. However, this type of construction is still not fully exploited as a result of the acknowledged difficulties involved in forming construction-efficient and cost-effective connections. Furthermore, there is a lack of information on the structural behavior of the cold-formed steel connections. In this study, the research on various cold-formed steel connections was comprehensively reviewed from both fundamental and structural points of view, based on the available experimental and analytical data. It reveals that the current design codes and guidelines for cold-formed steel connections tend to focus more on the individual bearing capacity of the fasteners rather than the overall structural behavior of the connections. Significant future work remains to be conducted on the structural performance of cold-formed steel connection. In addition, extensive previous research has been carried out to propose and evaluate an economical and efficient connection system that is obtained from the conventional connecting techniques used in the hot-rolled industry. These connecting techniques may not be suitable, however, as they have been adopted from hot-rolled steel portal frames due to the thinness of the sheet in cold-formed steels. The review demonstrates that with the increasing demand for cold-formed steel constructions throughout the world, it is crucial to develop an efficient connection system that can be prefabricated and easily assembled on site.

Keywords: connections; cold-formed steel; fundamental behavior; structural performance

1. Introduction

Cold-formed steel structural members are extensively used in building construction throughout the world due to their excellent architectural and structural merits, particularly in terms of their strength-to-weight ratio, stiffness, recyclability, fast construction speed, and aesthetic appearance. Despite the awareness of these advantages, this type of construction is still not being fully exploited due to the practical difficulties involved in forming appropriate cost-effective and construction-efficient connections and predicting precisely their behavior.

Efficient connections between cold-formed steel sections will not only reduce cost and

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construction time, but will also achieve practical and efficient structural systems (Yu *et al*. 2005). When analyzing the behavior of connections, the three main characteristics that need to be explored are strength, stiffness and ductility (Swanson and Leon 2000, Qin *et al*. 2014a, b, g). The types of connections can be classified by means of load transfer as mechanical interlocking, welded, or adhesive bonding connections (Pedreschi and Sinha 1996). The generally used fasteners for cold-formed steel are bolts, screws, welds, cold rivets, and other special devices such as metal stitching and adhesives. Press-joining, which to date has not been fully researched, is a recently developed technique used for connecting cold-formed steel sections.

The primary objective of this study is to review the research of a variety of cold-formed steel connections from both fundamental and structural point of view based on the available experimental and analytical studies, and to show that the performance of cold-formed steel connections is significantly different from the behavior of the connections used in hot-rolled steel construction due to the fact that the connected parts are usually much thinner in cold-formed steel construction.

2. Fundamental behavior

2.1 Bolted connections

Bolts with nuts are threaded fasteners, which are assembled in preformed holes through the material elements to be joined (Toma *et al*. 1993). Many bolted connections are considered to be much more flexible than their welded counterparts (Shen and Astaneh-Asl 1999) and thus, are widely used in cold-formed steel construction. Due to the thinness of the connected sheet, the structural performance of bolted connections in cold-formed steel construction is quite different from that in hot-rolled steel application. Excessive dishing of the connected sheets and bolt rotation can be observed in cold-formed steel bolted connections (Yu and Panyanouvong 2013). Deformations of a bolted connection of two cold-formed steel members is mainly due to the bearing deformation produced by the bolt into the thin sheet, the deformation of the connected members due to the local buckling of thin walls, as well as the bolt's slippage due to hole clearance (Dubina and Zaharia 1997). The progressive growth of deformation and the corresponding increase in flexibility leads to unacceptably large deformations in some cases, which eventually may lead to failure (Lim and Nethercot 2004a). Bolt-hole elongation is defined by Lim and Nethercot (2004a) as the term used to describe the deformation of the bolt-hole caused by bearing of bolt-shank against the bolt-hole and it depends on several factors: the thickness of the cold-formed steel plate; the diameter of the bolt-hole; the material properties of the cold-formed steel plate; the diameter of the bolt; the material properties of the bolt; and whether the bolt-shank is plain or threaded.

Differing from the hot-rolled steel construction that allow both slip-critical (also called friction-type) and bearing-type connections, cold-formed steel connections are designed as bearing-type connections only and bolts installed to a snug tight connection are normally adequate. Four distinct failure modes were observed in cold-formed steel bolted connections by Winter (1956a) at Cornell University and followed later by a number of researchers (Yu 1982, Zadanfarrokh and Bryan 1992) as shown in Fig. 1. Type I, shear failure of the sheet, which occurred for relatively short edge distance; Type II, bearing failure of the sheet, which occurred chiefly for longer edge distance; Type III, rupture in the net section of the sheet, which occurred

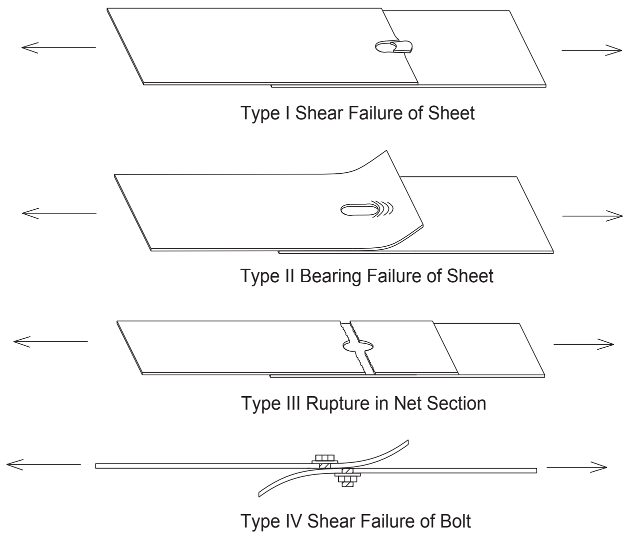


Fig. 1 Basic types of failures of thin sheet bolted connections (Yu and Panyanouvong 2013)

when the strength of the bolted connection is greater than the ultimate strength of the net section; Type IV, shear failure in the bolt, with more or less pronounced preceding elongation of the hole. These four types of failure modes provide the basis for the current design provisions in different codes of practice. However, previous research indicated that not all failures of the connections were of these clearly distinct types. A considerable number of connections failed in combined modes.

For relatively short end distance along the direction of the applied load, the connection may fail by longitudinal shearing of the steel sheet along two almost parallel lines, where the distance of separation is close to the diameter of the bolt (Fig. 1, Type I). It was discovered by several researchers (Winter 1956a, Zadanfarrokh and Bryan 1992) and they found that this failure mode largely depended on the thickness of sheet, the tensile strength of the connected sheet, and the end distance. It was further investigated and proved by Winter (1956b) that the longitudinal shearing of the sheet along the load direction was more likely to occur when the edge distance to bolt diameter ratio was less than 3.5. More conclusions included that the method proposed for common bolts was still applicable to high-strength bolts and the increase in bolt torque did not influence the ultimate failure load of the connection. The work by Dhalla *et al*. (1971) showed that for low ductility cold-formed steel, the shear strength was only about 15% lower than that for high ductility steel in terms of coupon tensile strength.

For sufficiently large bolt end distance, the connection may fail by bearing or piling up of steel sheet in front of the bolt (Fig. 1, Type II). General information on the bearing strength of bolted connections may be found in a number of specialist papers. Winter (1956a, b) of Cornell University initiated the research for 574 cold-formed steel single-bolt and two-bolt connections in the mid-1950s. The variables considered in his tests included bolt diameter, sheet thickness, edge distance, mechanical properties of sheets and bolts, etc. The research conducted at the University of Missouri-Rolla (Yu 1982) was one of the most recognizable works on this failure type and revealed that the hole extension prior to reaching the bearing strength was associated with an upper limit. The limitation of the connection movement was about 6.4 mm. In the classic work by

Zadanfarrokh and Bryan (1992), single shear lap connections, for which bolt tilt was the dominating influence, was the main consideration. These studies showed that the bearing capacity primarily depended on the thickness of the thinnest sheet, the tensile strength of the connected sheet, the type of bearing connection, the bolt diameter to sheet thickness ratio, the ultimate strength to yield strength ratio of the connected sheet, and rotation of fasteners. Rogers and Hancock (1998, 1999a, 2000) conducted bolted connection tests for estimating the prediction accuracy of current design standards for new structural steel materials G300 and G550. The test results indicated that the connection provisions set out in the AS/NZS 4600, AISI and Eurocode cold-formed steel design standards could not be used to accurately predict the failure mode of bolted connections that were fabricated from thin G300 and G550 sheet steels. In addition, these design codes could not be used to accurately determine the bearing capacity of bolted connections based on a failure criterion for predicted loads. A gradated bearing coefficient method, which was dependent on the thickness of the connected materials and the size of bolts used in the connection, was developed based on the measured variation in bearing resistance. It was recommended that the graded bearing coefficient formulation, the unreduced net section resistance, and the Eurocode design method for end pull-out be used in the design of bolted connections. Meanwhile, the presence of washers is significant when bearing is the failure mode (Wallace and Schuster 2002). It was found that the maximum bearing strength for bolted connection with washers was approximately 45% larger than that without washers (Chong and Matlock 1975, Gilchrist and Chong 1979). Moreover, a value of 2.5 for the edge distance to bolt diameter ratio was appropriate to distinguish the bearing of sheets from the longitudinal shearing of sheets. The research by Wallace *et al*. (2001a, b) and Wallace and Schuster (2002) was adopted to extend the Rogers and Hancock equation in AISI to utilize a function of bearing factor to distinguish between connections with and without washers when bearing failure occurred. Based on research at the University of Missouri-Rolla (LaBoube and Yu 1996), the effect of the presence of hole elongation prior to reaching the limited bearing strength of a bolted connection was recognized. The researchers adopted an elongation of 6.4 mm as the acceptable deformation limit, which was consistent with the permitted elongation prescribed for hot-rolled steel. Yu's group (Yu *et al*. 2011, Yu and Panyanouvong 2013, Yu and Xu 2013) presented experimental research to study the bearing strength of bolted connections with a gap and using oversized holes, which was not fully studied by previous researchers. AISI S100 indicates that slotted holes can be used in cold-formed steel bolted connections. However, the length of slotted holes should be normal to the direction of the shear load and washers or backup plates should be applied to outer plies for better behavior.

The load capacity of a bolted connection with respect to net section rupture (Fig. 1, Type III) has been shown to be influenced by several factors. Tests conducted by Winter (1956a) used connections under bolt head and nut to alleviate the stress concentration for the low ductility steel. Research by Yu (1982) found that the allowable tension strength for the net section of connected sheets was determined by the tensile strength of the connected part instead of the yield strength of steel and was also based on the type of connection, either a single shear lap connection or a double shear lap connection. In 1992, Zadoanfarrokh and Bryan (1992) also investigated the shear bearing failure in connection sheets. Besides observing the Type III failure of tearing of sheet in the net section, the effects of bolt diameter to bolt spacing perpendicular to load line ratio on the tensile strength of bolted connections with washers were found as well. Currently, the net section tensile strength is specified in the North American Specification for the Design of Cold-formed Steel Structural Members 2012 (AISI S100-2012) and in the Australian code AS/NZS 4600:2005 (AS/NZS 4600: 2005). It cannot be simply calculated by multiplying the net section area by the material tensile strength, which would otherwise imply a full net section efficiency. AISI S100 and AS/NZS 4600 incorporate simple formulae and constant reduction factors, respectively, for determining the net section tensile strength. The design equation specified in AISI S100 was recommended by LaBoube and Yu (1996) according to their test results and the net section efficiency factors are functions of the eccentricity and the length of connection only. On the other hand, the net section efficiency factor in the design equation specified in the Australian code is a constant depending on the presence of washers and whether the connection is single or back-to-back. Maiola *et al*. (2002) and Pan (2004) found the theoretical predictions computed by the design equations specified in both codes to be overestimated compared to the experimental results. Pan (2004) proposed an empirical equation as a function of the connection eccentricity, the connection length, the web depth, and the flange width. Regression analyses of test results are also popular in the literature (Paula *et al*. 2008, Prabha *et al*. 2011, Salih *et al*. 2013). However, design equations derived from regression analysis have been shown by Teh and Gilbert (2012, 2013a, b) to have pitfalls if not handled properly. Teh and Gilbert pointed out the net section efficiency of a cold-formed steel channel brace bolted at the web was affected by three distinct factors: the in-plane shear lag associated with stress concentration around a bolt hole that is also present in flat sheets, the out-of-plane shear lag that is also present in an I-section bolted at the flanges only, and the bending moment arising from the connection eccentricity with respect to the neutral axis. The equation proposed by Teh and Gilbert (2013a) for a channel brace bolted at the web was modified by Teh and Gilbert (2013b) to suit an angle brace bolted at one leg. This three-factor approach was further affirmed by Teh and Yazici (2013) based on the test results of 55 single and back-to-back channel braces bolted at the web.

The shear failure of bolts (Fig. 1, Type IV) is the type of failure by shearing of the bolts that occurs at the strength equal to 0.6 times the tensile strength of the bolt (Yu 2000). A number of single-shear and double-shear tests have been performed at Cornell University in the 1950s to study the type of failure caused by shearing of the bolt (Winter 1956a, b). It was found that the shear-to- tension strength ratio is independent of the bolt diameter, and the ratios are equal to about 0.72 and 0.62 for single-shear and double-shear test, respectively.

Chung and Ip (2001) established a finite element model with three-dimensional solid elements to investigate the bearing failure of cold-formed steel bolted connections under shear. Stress-strain curves, contact stiffness and friction coefficients between element interfaces, and clamping forces developed in bolt shanks were regarded as important parameters for the accurate prediction of the deformation characteristics of bolted connections (Chung and Ip 2000). It was found that the design rules were not applicable for bolted connections with high strength steels due to reduced ductility. A semi-empirical design formula for bearing resistance of bolted connections, applicable for both low strength and high strength steels with different ductility limits, was proposed. A strength coefficient was utilized to relate the bearing resistance with the design yield and tensile strengths of steel strip. It should be noted that initial geometric imperfection in thin-walled members should be considered in FE modeling in order to simulate the actual shape of specimens. Kim *et al*. (2008a) investigated the influence of initial geometric imperfection on the ultimate behavior of cold-formed steel bolted connections. A shell element with initial imperfection in the FE modeling of bolted connections was introduced so as to obtain time efficient analysis results and induce the curling behavior observed from both experimental results and FEA results of solid element models with no initial imperfection. Bolted connections with thin-walled steel were likely to involve curling as end distance grows larger. It was also believed that curling was caused by the local buckling of plate in the compressive stress field of the bearing part of the connection. Winter

(1956a) and Rogers and Hancock (1998) pointed out the curling behavior in their test results. Furthermore, experimental and numerical investigations on bolted connection performed by Chung and Ip (2000, 2001) showed curling of the plate end. Nevertheless, it was thought that curling had a negligible impact on the ultimate strength of bolted connections and thus, the curling was not taken into account in estimating the ultimate strength. Research by Kim and his cooperators (Kim and Han 2007, Kim *et al*. 2008b, 2011, Lim *et al*. 2013) showed that out-of-plane curling occurred in bolted connections with a relatively large end distance or edge distance and led to strength reductions. Modified strength equations for considering the strength reduction accompanied with curling were proposed for plate thickness ranged from 1.5 to 6.0 mm, end distance or edge distance ranged from 12 to 60 mm, pitch and gage of 30 mm, and bolt diameter of 12 mm.

2.2 Screw connections

Self-drilling screws are also a primary means of fastening cold-formed steel members in light-gauge steel constructions including domestic, agricultural, and industrial structures. The self-drilling screws drill their own hole and form their mating thread in one operation (Toma *et al*. 1993). The typical failure modes associated with screw connections include shearing of the screw, edge tearing, tilting and subsequent pull-out of the screw, and bearing failure of the connected members (Yu 2000). Fan *et al*. (1997a) pointed out that the screw threads prevent the screw from being moved along its axial direction but have little effect on preventing rotation. Frequently, the screws do not fail in shear, but rather experience rotation resulting in the development of a tensile component of force along the length of the fastener. As the rotation takes place, bearing causes the hole to become larger, rendering the threads less effective and causing the fastener to pull out. With decreasing steel thickness, tilting becomes the predominant failure mechanism while bearing stress becomes more critical with increasing steel thickness (Dawe and Wood 2006).

Fasteners installed in practice often suffer from a variety of in situ conditions that can affect the capacity of the fastener, such as construction tolerances creating combined shear and bending in the fastener (rather than pure shear), overtorqueing the fastener which can cause stripping of the thread or shearing off the head, nonperpendicular insertion of the fastener and accidental impact on the structure creating dynamic loads on the fastener (Bambach and Rasmussen 2007). Past research into the behavior of screw connections has typically been in the form of single lap tests in which the lap splice is subjected to a tensile force, resulting in the shear of the screws. Pekoz (1990) first recommended a set of design equations for estimating the strength of cold-formed steel-to-steel screw connections. These equations were reported to be formulated from an analysis of 3500 plus connection test data points.

Work by Fan *et al*. (1997a, b) dealt with numerical simulation related to single lap arrangements. Rogers and Hancock (1999b) summarized the results detailing the behavior of thin G550 and G300 sheet steel, single overlap, screwed connections and evaluated the accuracy of different design standards. Mahendran and Mahaarachchi (2002) studied the pull-out strength of screwed connections in steel roof and wall cladding systems between thin steel battens made of different thickness and steel grades, and screw fasteners with varying diameter and pitch. Simple design equations were proposed to take into account the strength reduction caused by fluctuating wind loading. A study, performed at the University of Missouri-Rolla involving testing 200 single lap screw connections (LaBoube and Sokol 2002), assessed the influence of design parameters typically employed in residential construction such as fastener patterns, screw spacing, stripped screws, and the number of screws on connection strength. The research demonstrated that the screw pattern did not significantly influence the strength of the connection. However, the assumption that the connection strength for a multiple screw connection is proportional to the number of screws in the connection pattern was found to be unconservative. When sheet thickness and screw size are held constant, increasing the number of screws results in a decrease in strength per screw. This is a phenomenon known as the "group effect." The group effect is the strength per screw of a given connection divided by the connection strength for a single screw connection with the same sheet thickness. In other words, if all screws were to act equally, the group effect would be 1.0. Also, connections formed with screws having damaged threads ("stripped") perform as well in shear as connections without stripped screws. Serrette and Peyton (2009), based on a survey of published manufacturer screw values, noted that as the thickness or yield strength of the connected elements/components is increased, the ratio of the fastener strength to the bearing/tilting strength decreases to values less than one-half in some cases. On this basis, it was recommended that the AISI S100 specification be revised to specify limits on manufacturer computed safety and resistance factors.

2.3 Welded connections

The typical welded connections in cold-formed steel structures are cold-formed tubular connections. Structural hollow sections (SHS) are widely used in structural components due to their merits such as good structural properties and esthetic appearances (Qin *et al*. 2014 f). Many experimental tests (Qin *et al*. 2014 e) and numerical analysis (Qin *et al*. 2014 c, d) on the capacity of hot rolled structural hollow section connections have been performed. However, only limited test data are available for cold-formed structural hollow section connections.

Zhao (2000) studied the deformation limit and the ultimate strength of welded T-joints in cold-formed RHS sections. Both web buckling failure mode and chord flange failure mode were investigated. Zhao and his colleagues (Mashiri *et al*. 2000, 2002; Mashiri and Zhao 2004) adopted boundary element method and plastic line method, respectively, to analyze the crack propagation and the strength of the welded connections. Teh and his co-workers (Teh and Hancock 2005; Teh and Rasmussen 2007), and Landolfo *et al*. (2009) conducted extensive tests to evaluate the strength of the welded cold-formed steel connections. A summary of the experimental investigation of the static strength of welded tubular T-joints made of cold-formed carbon steel hollow sections can also be found in Feng and Young (2008). The deformation, ultimate load capacity, and fracture behavior of full-scale welded K-joints fabricated from cold-formed rectangular hollow sections have been studied both numerically and experimentally by Björk *et al*. (2006). An extensive assessment (Björk *et al*. 2008) revealed that crack advance, once initiated, would be expected to continue at a nearly constant load.

2.4 Press-joining

A relatively new technique for joining cold-formed steel members is press-joining where a male and female punch and die are used to shear the material in the connection, which is then pressed together to form a clinched connection (Hancock 1997). The process of press-joining is clean, energy efficient and formed in a single step operation that does not destroy the protective coating of steel (Pedreschi and Sinha 1996). A detailed investigation was conducted by Pedreshi *et al*. (1997) on the potential application of this technique to cold-formed steel structures. Press-joining can provide sufficient strength to be used in cold-formed steel applications such as lightweight steel-framed wall panels or lightweight steel trusses (Makelainen and Kesti 1999).

2.5 Connection behavior at elevated temperatures

Some research has been carried out in cold-formed steel connections at elevated temperatures. Bolted connections between cold-formed steel members at elevated temperatures (Lim and Young 2007) were studied to evaluate the reduced moment-capacity of the channel-sections at the joints and the reduced shear strength in bearing of the bolt-hole, which in fire will be required to resist catenary action. Chen and Young (2007) numerically investigated cold-formed steel lipped channel columns with the consideration of initial local and overall geometrical imperfections at elevated temperatures. It is shown that the effective width and direct strength methods are able to predict the cold-formed steel lipped channel column strengths at elevated temperatures. The research on screwed connections of thin sheet steels at elevated temperatures was conducted by Lu *et al*. (2011) and Yan and Young (2012). Several failure modes were observed including the bearing, tilting and bearing, screw shear, net section tension and material failure. It is indicated that the strengths of the single shear screwed connections predicted by the current specifications are generally conservative at elevated temperatures.

3. Structural behavior

Various modern codes of practice for the design of cold-formed steel structures (AISI S100-2012 2012, BS EN 1993-1-3 2006, AS/NZS 4600:2005 2005) incorporate a number of design methods for the evaluation of section properties and member resistances of typical sections against local buckling, distortional buckling, as well as global buckling. A comparative study of different connections in cold-formed steel can also be found (Lennon *et al*. 1999). However, it should be noted that for the design of connections, only rules for determining the strength of individual fastenings such as welds, bolts, and screws are provided. There is a comparative lack of information on the structural behavior of the connections between cold-formed steel members. Structural connections, which play an important role in the design and cost of the structural system, have become a significant topic for research in the cold-formed steel industry. This section reviews in detail the research on connections used for various types of systems.

3.1 Beam-to-column connections

Connections in multi-story buildings comprised of gusset plates, through plates and channel beams connected to columns were tested under monotonic and cyclic loading. Cold-formed steel elements with conventional types of beam–column moment resistant connections (commonly used with hot-rolled steel sections) were tested by Valencia-Clement (2009). The results show that they failed to provide sufficient ductility needed for seismic design. Extensive studies of the structural behavior of connections made from cold-formed members have been conducted in Hong Kong by Chung and his co-workers (Chung and Shi 1998, Chung and Lau 1999, Chung and Lawson 2000, Wong and Chung 2002, Chung *et al*. 2005). Their contributions in the area of connections have concentrated on the behavior of beam-to-column and beam-to-beam bolted moment connections intended to transmit large moments and have been restricted to rational considerations of design strength. The basic configuration of bolted moment connections among cold formed steel members in their research is as follows: structural members such as columns and beams are formed with two lipped C-sections back-to-back with interconnections at regular intervals. Moment connections between main beams and columns (i.e., primary beam column connections) are formed with gusset plates (either hot-rolled or CFS). Moment connections between secondary beams and columns (i.e., secondary beam column connections) are formed with folded angles. Only the webs of lipped C sections are bolted onto gusset plates and folded angles; the flanges are not connected for ease of construction. Two to four bolts per member may be used at the connection. Uang *et al*. (2010) and Sato and Uang (2010) proposed one type of moment frame that is composed of cold-formed hollow structural section column and C-section beams interconnected by snug-tight high-strength bolts. Good ductility and high moment resistance were achieved through bolt slip and bearing in the connection. Based on both the experimental and numerical results, an equivalent lateral force design procedure for the seismic design of a special bolted moment frame (SBMF) has been developed and incorporated (Sato and Uang 2009) into the AISI seismic standard (AISI S110-07 2007). Recently, Sabbagh and his research group (Sabbagh *et al*. 2010, 2011, 2012a, b, 2013) conducted extensive experiments and FE analyses to develop a ductile CFS moment resistant beam-to-column connection. The beam section is formed with curved flanges which increase local buckling resistance and avoid the large flat flange width/thickness ratios found in conventional CFS beam sections such as back-to-back channel, and lipped or unlipped sections. The curved flange section has been arrived at through a step-by-step process of increasing the number of flange bends in the section. The through plates are the main components of the connection, transferring the forces to both near and far sides of the column by in-plane action. Sets of out-of-plane stiffeners are also needed to further improve the ductility and energy dissipation capacity of the beam–column assembly.

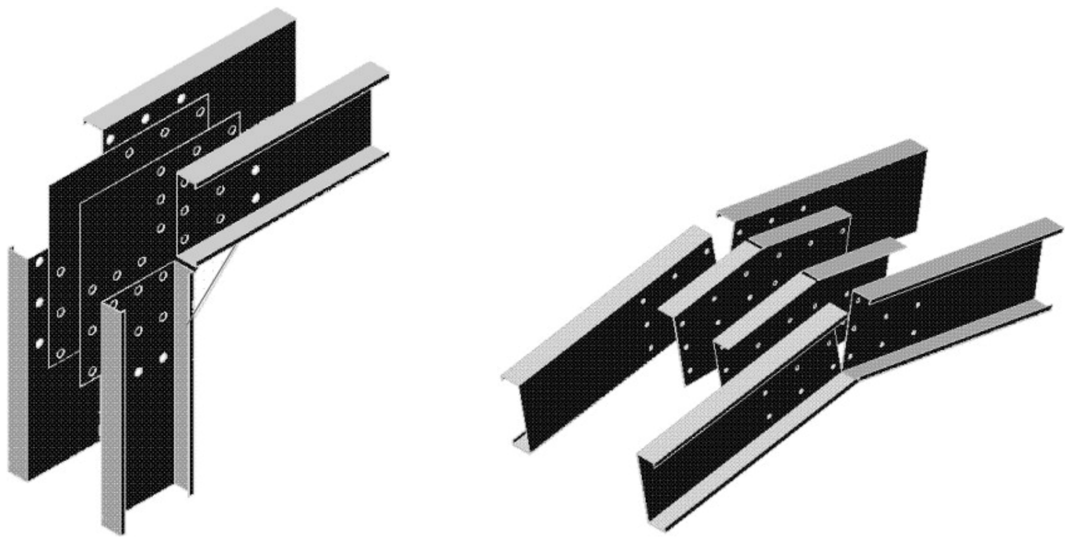
3.2 Portal frame connections

Wilkinson and Hancock (2000) analyzed various types of knee joints for steel portal frames constructed from cold-formed rectangular hollow sections. The different tested connection types included welded stiffened and unstiffened knee connections, bolted knee connections with end plates, and connections with a fabricated internal sleeve. It was found that, under opening moment, fracture often occurred in the heat-affected zone of the cold-formed rectangular hollow sections at low level of rotation. Under closing moment, web local buckling occurred near the connection. The study aimed at developing a connection that was able to form a plastic hinge, allowing for the plastic design of portal frames using rectangular hollow sections. The results indicated that the internal sleeve connections exhibited the most suitable behavior for plastic design.

A series of tests were carried out on various conventional knee connections such as knee-braced connections, mitered connections, bolted end-plate connections with various web and flange stiffening configurations for cold-formed channel portal frames at the University of South Australia (Mills 2000, Mills and LaBoube 2004). The study revealed that the connections did neither achieve the moment capacity required for typical portal frame nor the section moment capacity of the component cold-formed channel. Thus, the connection capacity would not allow the full capacity of the channel to be used in the portal frame design. Based on this observation, an alternative knee connection was developed using self-drilling screws. The test results reinforced the conclusion that the bolted end-plate moment connection was not suitable for application in the knee connections of portal frames. The self-drilling screw connections on the other hand, were simply to design and showed excellent moment capacity and rigidity.

Kwon *et al*. (2006) conducted several connection tests which were composed of closed cold-formed steel sections for a pitched roof portal frame. The connection test specimens consisted of column base, eave, and apex connections of the portal frame. The main factors considered in the tests were the connection element shape, the thickness of the connecting members, and the restraint against torsion of the rafter section. The test results were compared with the nonlinear analysis obtained from the program LUSAS. It was observed that the failure of the connections of portal frames was caused by the local buckling at the cold-formed steel section and the excessive rotation of connections, followed by the rupture of clinching. The proposed connections exhibited sufficient strength and stiffness to effectively constitute the pitched roof portal frame. The pitched roof portal frame test indicated that the semi-rigid connections had excellent structural performance and could be applied to portal frames successfully.

Lim and Nethercot (2002) described a cold-formed steel portal framing system in which back-to-back cold-formed steel channel-sections were used for the column and rafter members and brackets, and bolts between the webs of the channel-sections were used to form the connection, as shown in Fig. 2. They utilized both non-linear large-displacement elasto-plastic finite element analysis and a linear beam idealization (Lim and Nethercot 2004b) to provide design recommendations for the eaves bracket of a cold-formed steel portal frame. Two potential buckling failure modes were observed using parametric studies: buckling of the stiffened free-edge into one-half sine wave and local plate buckling of the exposed triangular bracket area. In addition, considering the semi-rigid behavior of these connections, rational values of deflection limits appropriate for use with cold-formed steel portal frames were recommended (Lim and Nethercot 2003b). Lim and Nethercot (2003a) further noted that the moment-capacity of these connections was lower than that of the cold-formed steel sections being connected due to the web buckling, caused by the concentration of load transfer from the bolts and accompanied by a slight distortion of the flanges. This form of failure was first observed by Kirk (1986) and was then discussed in the research conducted by Chung and Lau (1999), and Wong and Chung (2002) at the Hong Kong Polytechnic University.



(a) Eaves connection (b) Apex connection

Fig. 2 Details of the arrangement for the eaves and apex connections (Lim and Nethercot 2002)

The eaves connections of cold-formed steel portal frames constructed from single channels connected back-to-back using bolts were studied by Dundu and Kemp (2006a, b). The major objective of this research was to evaluate the performance of connections in portal frames applied for small span building construction. The variables in the tests were the number of bolts in the connection, the points of contraflexure, the width of the channel flanges and the strength of the channels. The observed failure modes included local buckling of the compression zone of the flange and web of the channels, lateral torsional buckling of the channels between points of lateral support, and bolts in bearing. It was reported that the final failure mode in all specimens was local buckling of the compression flange and web. Considerable ductility was achieved in the back-to-back bolted connections, which should be sufficient to accommodate plastic analysis of the portal frames.

3.3 Stud-to-track connections

Cold-formed steel stud wall systems are constructed with a combination of stud and track sections. The lateral strength and performance of stud-to-track connections has been investigated by a number of researchers.

Drysdale and Breton (1991) at Macmaster University conducted 109 stud-to-track connection tests as part of a larger study on the performance of steel stud brick veneer wall assemblies. The parameters considered included the size and thickness of the steel stud and track, the number of screws used to make the connection and the gap between the end of the steel stud and the inside face of the track. It was found that the type of fastening detail significantly affected the stiffness of

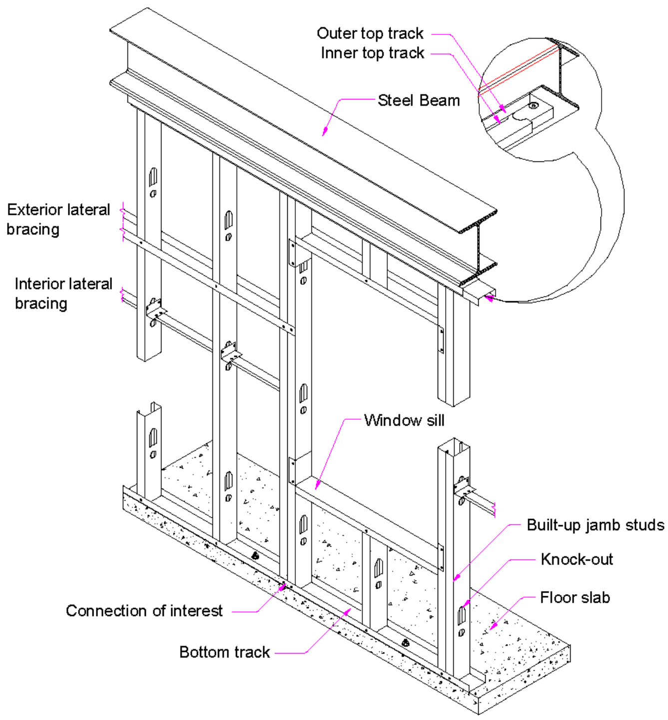


Fig. 3 Typical infill wall application (Lewis 2008)

the connection. The typical failure mode was web crippling of the stud due to the fact that the steel stud transfers wind load to the supporting top and bottom tracks and, when the resulting end reactions are large enough, the local stress concentrations lead to web crippling. In some cases, the specimens using clip angles to connect the web of the stud to the track failed not by web crippling but shearing of one of two bolts connecting the clip angle to the stud or pulling the nail anchors out of the supporting concrete beam.

Fox and Schuster at University of Waterloo (2000) presented a design procedure to calculate the lateral capacity of the stud-to-track connections. Two failure modes were recognized in the tests: web crippling of the stud and punch-through of the track. The proposed method could be applied to 92 mm and 152 mm stud and track with the thickness ranging from 0.84 mm to 1.91 mm. The web crippling predictor equation was also reported for a gap between the stud and track of up to 12 mm.

Lewis (2008) investigated the connection between built-up stud members and the bottom track, both at interior locations, as shown in Fig. 3, and at end locations, such as would be found at a doorway or the corner of a building. A total of 94 tests were conducted with the parameters covering different depths of stud and track, thickness of stud and track, configuration of jamb studs, location of jamb studs in the track, screw size, screw location, and yield strength of steel. It was shown that a single screw in the top flange would preclude web crippling or track punch-through failure and result in excessive deformation of the track flange. A single screw in the bottom flange would behave in a similar manner as a connection with two screws, as long as the screw was large enough not to fail first. Web crippling coefficients were developed based on a regression analysis of the test data.

3.4 Purlin connections

Lapped connections are used in practice to connect purlins over their internal support. Various FEA and experimental works have been conducted to investigate the strength and stiffness of the lapped connections as well as the nonlinear behavior and failure modes of the connections with different configurations (Ghosn and Sinno 1995, Ghosn 2002, Ho and Chung 2004, 2006a b, Chung and Ho 2005, Zhang and Tong 2008, Dubina and Ungureanu 2010). An experimental investigation was reported that showed the typical failure modes and stress distribution patterns of cold-formed steel Z beams with lap connections (Ghosn and Sinno 1995). The investigation indicated that the failure was governed by the bending stress and the failure mode was related to the length ratio between lap and beam span. A similar subject was studied and reported in several papers (Ho and Chung 2004, 2006b). The influence of the ratios of lap length to section depth and to beam span length on the connection behavior was especially examined. An analytical method to determine the strength of Z sections with lap connections can be found in the paper by Chung and Ho (2005). Another experimental study for the behaviour of lapped Z connections was presented by Zhang and Tong (2008) by considering the effect of the use of self-drilling screws and slotted bolt holes. The test results presented in (Ho and Chung 2004, Zhang and Tong 2008) were further examined by Dubina and Ungureanu (2010), who suggested that the failure of lapped Z connections was primarily a result of combined bending and web crippling, rather than the combined action of bending and shearing as indicated in (Ho and Chung 2004, Zhang and Tong 2008).

Sleeve connections are also commonly used for purlin connection systems in modern roof construction. In comparison with lapped connections, literature surveys have indicated a scarcity of research on sleeve connections. One paper published two decades ago (Moore 1990) reported an experimental study on the moment-rotation response of three types of cold-formed steel sections (Zed, sigma and Zeta). It stated that the sleeve connection exhibits a highly non-linear rotational behavior. In recognizing the fact that rotational behavior is difficult to quantify precisely, the paper proposed simplified methods for determining the value of rotational stiffness in a secant term and the moment resistance. The proposed method employed an ideal elastic–plastic moment–rotation model to reproduce the results observed from tests. In the model, the rotation at ultimate moment was chosen as 0.05 radians. Nevertheless, with only three test specimens, results were not sufficient to establish a design formula to calculate the rotational stiffness or the moment resistance for sleeve connections. Another study was published on the sleeve connections for Z sections (Gutierrez *et al*. 2011). One of the conclusions from this research was that the sleeve connection has a semi-rigid characteristic in terms of the rotational stiffness and only provides a partial moment resistance. A finite element model was presented to simulate the connection behavior by using explicit dynamic analysis due to a high nonlinearity exhibited by this type of connection. Work by Yang and Liu (2012) assessed the moment–rotation behavior as well as the moment resistance of this type of connection. Engineering models were also developed to predict the connection behavior.

3.5 Shear panel connections

Studies on cold-formed steel shear wall sheathed with gypsum or wood based boards by Fulop and Dubina (2004), Della Corte *et al*. (2006), Branston *et al*. (2006) and Serrette *et al*. (1997) showed that the failure was mostly influenced by the failure of screws connecting the board to the cold-formed steel stud if the stud and its connections were adequately designed. Most of the non-linearity in the load-deformation response of the cold-formed steel shear wall panel under in-plane shear was due to the non-linear behavior of the screw connection between the boards and the cold-formed steel members. The failure of the screw connection in the CFS shear wall panels was due to the combination of the tilting of the screws in the plane perpendicular to the stud flange, bearing against the sheathing and complete screw head pull through the sheathing and edge tearing failure of sheathing.

Gypsum-based panels connected to cold-formed steel members were experimentally investigated by Miller and Pekoz (1994), Serrette *et al*. (1997), and Fiorino *et al*. (2007). A few experimental investigations have been conducted at lap connections between steel to steel sheathing (Fulop and Dubina 2006, Serrette and Peyton 2009) and steel to plywood/OSB board (Gad 1997) to understand the shear behavior of the screw and its failure modes. These studies tested screws with the shear force acting perpendicular to the free edge of the sheathing, while in a wall panel under in-plane shear the screws experience shear, essentially parallel to the sheathing edge. Nithyadharan and Kalyanaraman (2011) reported a more representative experimental setup to properly model the load-deformation behavior and strength. The shear resistance of cold-formed steel framed shear walls using thin sheet steels was investigated by Yu (2010). The failures of screws such as screw tilting and screw pullout were observed in the tests. The local pull-through failure of screwed connections in crest-fixed steel claddings was experimentally and numerically investigated by Mahaarachchi and Mahendran (2009). Self-tapping screws could also be used as connectors between adjacent wall panels and between panels and foundations in wooden buildings (Fragiacomo *et al*. 2011). The ductility in timber structures was studied by Jorissen and Fragiacomo (2011) and Bruhl *et al*. (2011). It was found that full threaded self-drilling screws

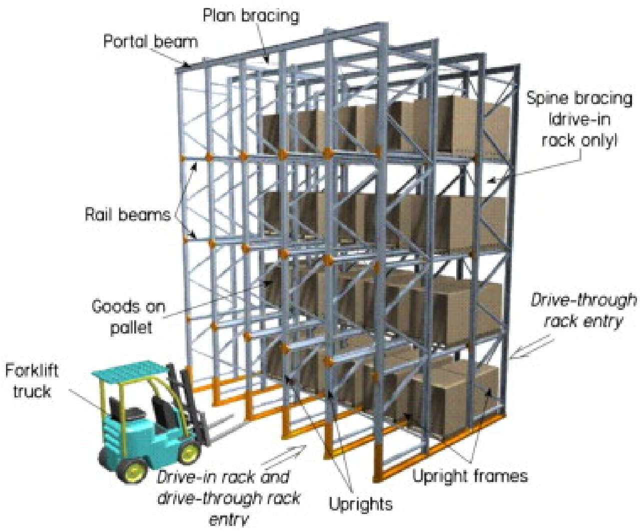


Fig. 4 Drive-in and drive-through racks (Gilbert and Rasmussen 2010)

could be used to prevent timber connections from splitting and improve the structural ductility for timber connections.

3.6 Rack connections

A few investigations were reported on cold-formed rack connections, mostly in European countries. Lewis (1991) provided a simple design approach to the stability of pallet rack structures and the effect that the moment-rotation characteristic has on the type of stability exhibited by the system. An experimental study was presented by Bernuzzi and Castiglioni (2001) on the behavior of connections in steel storage pallet racks. Eleven tests were executed on two different types of commercial products. The monotonic tests showed that the nodal zones were generally characterized by a satisfactory ductile behavior. The cyclic tests indicated that the connection behavior played a significant role on the overall frame performance. Bajoria and Talikoti (2006) conducted tests to determine the flexibility of beam-to-column connectors used in conventional pallet racking systems by cantilever and double cantilever test set-ups. The double cantilever connection was reported to be far superior to the conventional single cantilever connection. Prabha *et al*. (2010) carried out eighteen experiments on a commercially available pallet rack connection by varying parameters the most influencing the behavior such as the thickness of the column, depth of the connector and the depth of the beam. The flexibility of the connections was quantified and a general Frye-Morris type/three parameter power model type moment versus relative rotation relationship was developed. Gilbert and Rasmussen (2010) reported cyclic tests on bolted moment beam to upright connections of drive-in and drive-through storage racks as illustrated in Fig. 4. They found significant looseness in the connection stiffness after applying a non-negligible moment to the connection. Methods to design this type of rack connection were proposed based on the experimental observation.

4. Conclusions

In this study, the research on the cold-formed steel connections was reviewed comprehensively on the basis of the available experimental and analytical studies from both the fundamental and structural points of view. The review of the available publications reveals that extensive studies have been conducted to analyze and design efficient connection systems. The following conclusions can be drawn.

(1) The design codes and specifications for cold-formed steel connections tend to focus more on the individual bearing capacity of the fasteners rather than the overall structural behavior of the connections. While substantial information on the structural behavior of the connections between cold-formed steel members is available, significant future work remains.
(2) A large number of investigations have been carried out to design an economical and efficient connection system that deviates from the conventional connection techniques used in the hot-rolled industry. These connection techniques may not be suitable, however, as they have been adopted from hot-rolled steel portal frames due to the thinness of the sheet in cold-formed steel construction.
(3) With the increasing demand for cold-formed steel constructions in light agricultural, industrial and residential buildings, it is crucial to develop efficient connection systems that can be prefabricated and easily assembled on site.

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