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# Connection in Joints for Thin-Walled Steel Sections and Sheeting

*Thin-walled cold-formed members are thin, this will give rise to behavioural phenomena, which are not usually encountered in the more familiar hot-rolled sections. When compared to hot-rolled steel sections, cold-formed thin-walled steel sections are more likely to fail in local buckling, distortional buckling, various global buckling and shear buckling. This paper will discuss types of connection in Jjoints for cold-formed thin-walled sections and steel sheeting. Bolts, screws, blind rivets or cartridge fired pins are commonly used in joints for cold-formed thin-walled sections or steel sheet connections. Fasteners in light gauge steel tend to be relatively less stiff than their counterparts in heavier construction so that connection flexibility can be significant in certain assemblies. Furthermore, as in any load-bearing structure, it is important that connections are not brittle and this implies that there should be adequate deformation capacity.*

***Keywords***: *connection, joint, steel, cold formed, sheeting, sections*

## 1. Introduction

Bolts, screws, blind rivets or cartridge fired pins are commonly used in joints for cold-formed thin-walled sections or steel sheet connections. Fasteners in light gauge steel tend to be relatively less stiff than their counterparts in heavier construction so that connection flexibility can be significant in certain assemblies. Furthermore, as in any load-bearing structure, it is important that connections are not brittle and this implies that there should be adequate deformation capacity [1].

Failure of fasteners in shear can be one of the four types illustrated in Figure 1, namely shear of the fastener, crushing of the fastener, tilting and pull out of the fastener or yield in bearing. In tension, failure can be one of five types shown in Figure 2, i.e. tension failure of the fastener, pull out of the fastener, pull over of the sheeting, pull through of the sheeting or gross distortion of the sheeting.

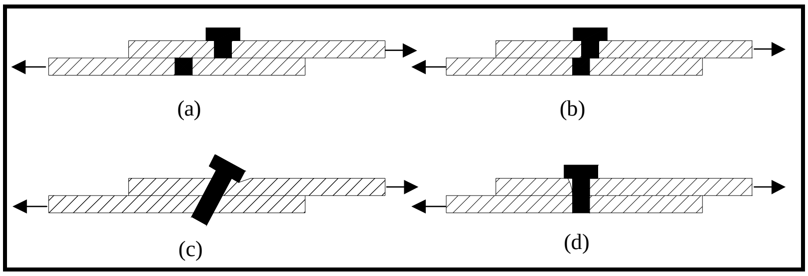


**Figure 1.** Failure modes for fasteners under shear force. (a) shear of the fastener itself; (b) crushing of the fastener; (c) tilting and pull out of fastener; (d) yield in bearing (tearing of the sheeting)

Since the thickness of cold-formed steel is very thin, some special techniques have been developed. Therefore, defining reliable values for design, which is based on tests, is a main problem for using different types of connections. Although BS5950 Part 5 does not give specific rules for the testing of fasteners of connections, the procedures in European Recommendation can be used in test to determine the strength and stiffness of fastener itself and the properties of connections incorporating one or more fasteners [2].

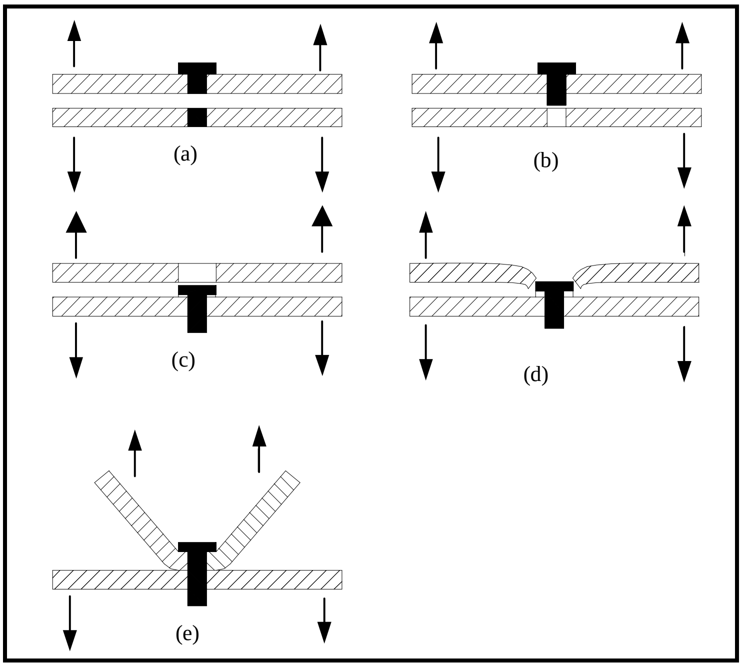
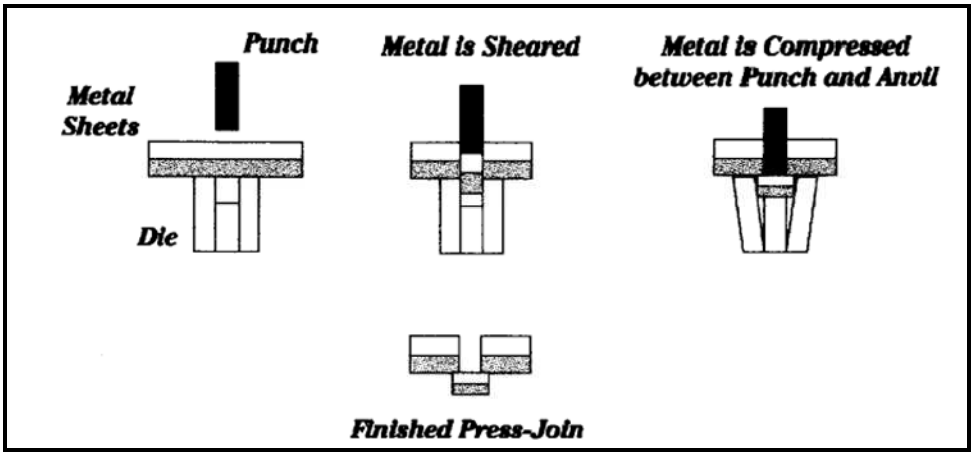


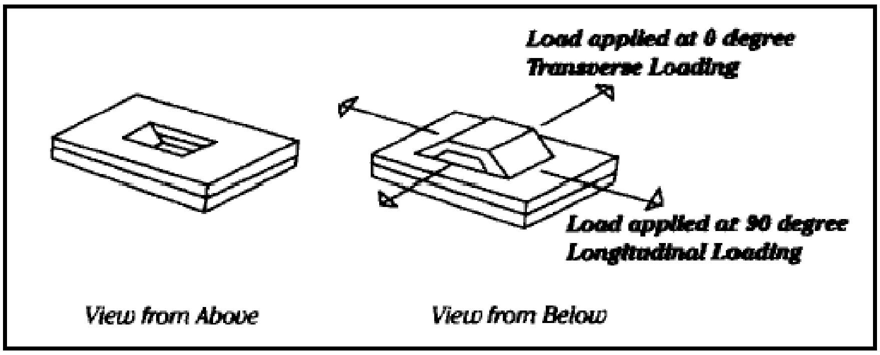
**Figure 2.** Failure modes for fasteners under tension force. (a) Tensile failure of the fastener itself; (b) pull out of the faster; (c) pull over of the sheeting; (d) pull through of the sheeting; (e) gross distortion of the sheeting.

Two new techniques, ‘press-joining’ and ‘rosette joint’, have recently been developed and studied. These two new techniques have several advantages over other common joining methods used in steel construction, such as bolting and welding. The processes of forming a press-joining joint uses a male and female punch and die to shear the material in the connection and then press together to form a clenched connection, as shown in Figure 3. The rosette joint is formed, using the parent material, in pairs of prefabricated holes by means of a special tool, as shown in Figure 4 [3]. Shear capacity of rosette joints is suitable for application in light-weight steel framed wall panels or light weight steel trusses [4].

A comparative investigation on the shear behaviour of mechanical connections in thin gauge steel. The connection techniques considered are press joining, Henrob fasteners, pop rivets and self-tapping screws. The shear tests have been carried out on rectangular press joining mechanical clinching connections, self-piercing rivet connections, pop rivet connection and screw connection in similar thickness of two layers of steel 1.0, 1.2, 1.6 and 2.9 mm thick [5]. They pointed out that in the small displacement range of loading press joining gave a non-linear load-displacement response while self-piercing rivets gave an approximately linear response [6]. Self-piercing rivets and press joining at 0°C orientation showed a high initial stiffness in comparison with the other types of connections. Self-piercing rivets showed a high peak load and high ductility as the rivet part of the connection linked the parent metal components [7].

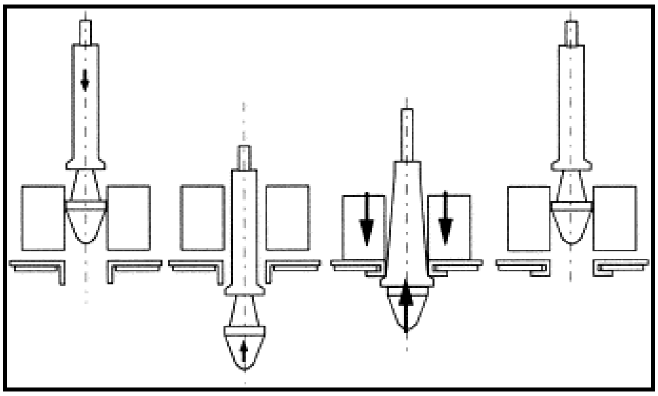


(a) Press-joining process

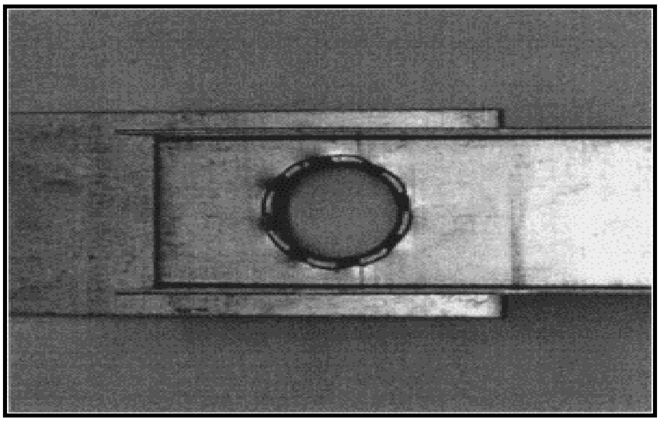


(b) Press-joining

**Figure 3.** A typical press-joining



(a) Rosette joining process



(b) Rosette joint

**Figure 4.** A typical rosette joint

The shear capacity of bolts given in BS5950 Part 5 (BSI 1998) and AISI (1996) is:

$$P_s = p_s A \tag{1}$$

where, A is the area of thread or shank in BS5950 Part 5 (BSI 1998) and the gross cross-section area of bolt in AISI (1996), $p_s$ is the shear strength of bolts.

The design bearing resistance per bolt for a connection regardless of the design standard used is as follows:

$$P_{bs} = Cdtp_y \tag{2}$$

where d is the nominal diameter of the bolt; t is the minimum thickness of the connected material; C is a bearing coefficient which is related to the thickness of the connected material, depending on whether there are washers under both the bolt head and nut. $p_y$ is the design strength of bolts in BS5950.

The bearing capacity of thin sheet steels is overestimated when the AISI and ENV1993-1-3 methods are used for bolted connection specimens that were composed of 0.80 and 1.00mm G550 and G300 sheet steels. They proposed a modification in the design method for bolted connections that are loaded in shear. The recommendation is that when using ENV1993-1-3, a graded bearing coefficient formulation in Equation (2), as given in Table 1, should be used. Rogers and Hancock (1999b) also reported the test results of single overlap screwed connections that were concentrically loaded in shear and fabricated with multiple-point fasteners using G550 and G300 sheet steels that ranged in base metal thickness from 0.42 to 1.00mm.

They found that the AISI design standards provided accurate load predictions for single-overlap screwed connections when the two joined sheet steels were of a similar thickness and failure was more likely to depend on tilting of the screws. However, when two different thickness sheet steels are connected with screws, failure will result from bearing distress in the thinner of the connect elements. The effect of different thickness between the two connected elements is to force the connection to fail in a combined bearing/tilting mode rather than a bearing mode as suggested in the AISI method.

To eliminate the wrong predicted result, they suggested using the graded bearing coefficient formulation, as shown in Table 2, to improve the accuracy of the predicted load resistance when two different-thickness sheet steels are joined. The minimum value obtained from equation (2) according to different thickness elements should be chosen as the ultimate design bearing resistance.

In all design methods, the nominal cross-section tension resistance of a connected material that is not subject to shear lag and fails by material yielding of the gross cross-section is formulated as follow:

$$P_n = A_n f_y \tag{3}$$

where $A_n$ is the net area of the section of the connected material. $f_y$ is the yield stress of connected material.

**Table 1.** Factor for bearing resistance of bolted connections

| d/t | C |
|---|---|
| $d/t \leq 10$ | 3.0 |
| $10 < d/t \leq 22$ | 4.0-0.1 d/t |
| $d/t \geq 22$ | 1.8 |

**Table 2.** Factor for bearing resistance of screwed connections

| d/t | C |
|---|---|
| $d/t \leq 6$ | 2.7 |
| $6 < d/t < 13$ | 3.3-0.1 d/t |
| $d/t \geq 13$ | 2.0 |

However, shear lag effects will always occur in tension connections using multi-bolts. When designing this type of connections, the effective net section, which is a function of the geometry of the connected member and the connection, should be used to account the shear lag effects. Research Shear lag effect can be considered by using a reduced net area [8]. The proposed formula for the effective net area is

$$A_{neff} = A_n (1.60 - 0.7\frac{A_n}{A_g})(1 - \frac{\overline{x}}{L}) \tag{4}$$

in which, $A_n$ is the net section area, which can be determined in accordance with current design codes; $A_g$ is the gross area of the connected member; $\overline{x}$ is the distance from the contact face of the connected member to the centre of gravity of the member, as shown in Figure 5; and L is the length of the connection.

The prediction of ultimate load based on the failure mode is proposed by adding the ultimate strength of the critical section of the connected leg and the strength contributed by the critical section of the outstanding leg, which can be described as:

$$P_u = f_u A_n + \beta f_y A_g \tag{5}$$

where, $A_g$ is area of the outstanding leg (gross area); $A_n$ is net area of the connected leg at the critical section; $f_y$ is the yield strength of the material; $f_u$ is the ultimate tensile strength of the material and $\beta$ should be equal to 1.0 for members with four or more fasteners per line in the connection and 0.5 for members with three or two fasteners per line in the connection [9].

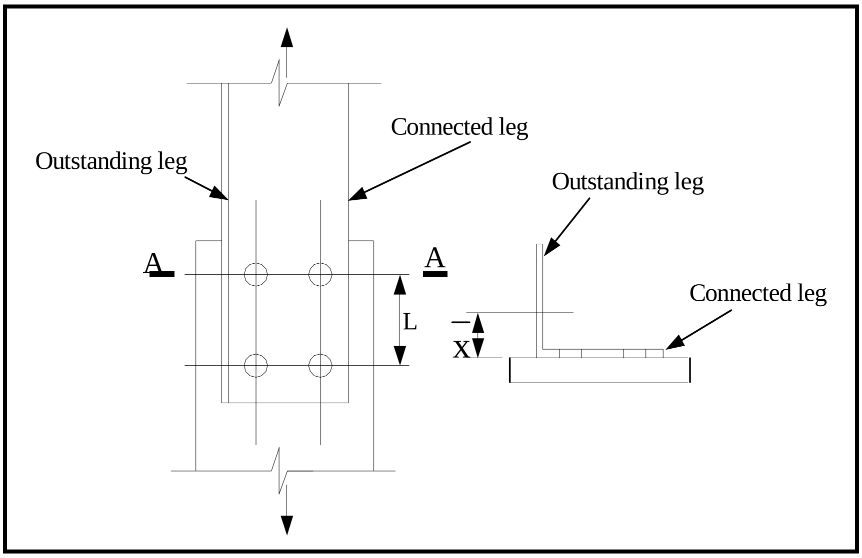


**Figure 5.** $\bar{x}$ and L magnitudes for bolted connections

The tensile strength of a C-shaped cold-formed steel section could be influenced by the shear lag and the cross-section is not fully effective. AISI predictions overestimate the capacity of this type of connections [10]. However, further tests and numerical investigation of the strength of other shapes of cold-formed steel section connected by bolts, which will be influenced by shear lag, should be done. The amended design equation in AISI specification supplement for the channel and angle sections under axial tension load is:

$$P_n = A_{neff} f_u \tag{6}$$

where, $f_u$ is the tensile strength of the connected part and $A_{neff}$ is the effective net area and can be calculated as:

$$A_{neff} = UA_n \tag{7}$$

in which U can be defined as follows:

For angle members having two or more bolts in the line of force,

$$0.4 \leq U = 1.0 - \frac{1.2\bar{x}}{L} < 0.9 \tag{8}$$

For channel members having two or more bolts in the line of force,

$$0.5 \leq U = 1.0 - \frac{0.36\bar{x}}{L} < 0.9 \tag{9}$$

where, $\bar{x}$ is the connection eccentricity (distance from shear plane to centroid of the cross-section) and L is the connection length.

In British Standard, the tensile capacity of a plain channel can be determined from:

$$P_t = A_{neff} p_y \tag{10}$$

where $p_y$ is the design strength and should be taken as the nominal yield strength but not great than 0.84 times the nominal ultimate tensile strength. $A_{neff}$ is the effective net area of the net section and can be calculated as:

$$A_{neff} = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)} \tag{11}$$

where, $a_1$ is the net sectional area of the connected leg and $a_2$ is the cross-section area of unconnected leg.

In AISI, separate requirements for the net cross-section tension resistance at connections where washers are provided under both the bolt head and nut have been given. The design equation for AISI is:

$$P_n = (1.0 - 0.9r + \frac{3rd}{s})A_n f_u \leq A_n f_u \tag{12}$$

where r is the ratio of the force transmitted by the bolt(s) at the section considered, divided by the tensile force in the member at that section; d is the diameter of the bolt(s); and s is the spacing of the bolts perpendicular to the line of the force or for a single bolt, the width of the sheet. The design formulation for ENV1993-1-3 is similar to that presented in (12). However, d is defined as the nominal diameter of the bolt hole.

End pull-out resistance of a bolted connection is dependent on the length of two parallel lines that extend from the bolt hole in the direction of the applied force. This type of failure differs from block shear rupture because each bolt tears our along its own path as shown in Figure 6. According to AISI, the nominal end pull-out resistance per bolt can be calculated as

$$P_f = tef_u \tag{13}$$

where e is the distance measured parallel to the direction of the applied force from the centre of a standard hole to the nearest edge of an adjacent hole or the end of the connected part; t is the thickness of the thinnest connected part and $F_u$ is the ultimate tensile strength of the connected part. The end pull-out resistance determined using ENV1993-1-3 is

$$P_f = tef_y \tag{14}$$

where $f_y$ is the yield stress of the connected part.

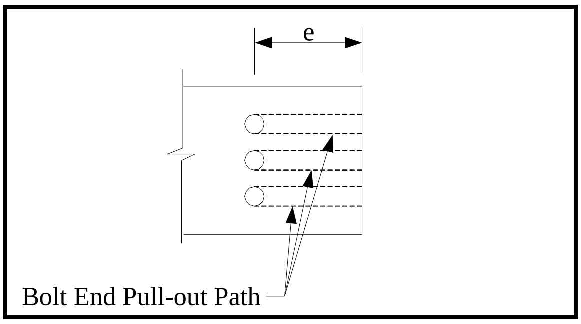


**Figure 6.** Individual Bolt End Pull-Out Path

The tension capacity of a bolt can be calculated as:

$$P_t = A_t p_t \tag{15}$$

Where, $p_t$ is the tension strength of the bolt, $A_t$ is the area of thread or shank in BS5950 and gross cross-sectional area of bolt in AISI. The tension capacity of bolt determined using ENV1993-1-3 is

$$P_t = 0.9 A_t f_{ub} \tag{16}$$

in which $f_{ub}$ is the ultimate tensile strength of the bolt material.

Until now, the database used to develop design methods for connections between cold-formed sections of sheets is mainly based on testing a very large number of connections. Because these tests are very expensive and take time, some numerical simulations using the finite element technique (Fan *et al* 1997) have been adopted to replace expensive tests. Fan *et al* (1997) pointed out that when performing computer modelling of a connection, attention must be paid to: large geometrical and material non-linearities in almost all the materials involved, the contact problems between materials, and the possible local fracture of the connected steel sheets in the screw vicinity at loading levels approaching the ultimate resistance of the connection.

## 5. Conclusions

A variety of joining methods is available for thin-walled structures. In comparison with thicker connections (t > 3 mm) the behaviour of connections in thin-walled elements is characterised by the small plates stiffness. Therefore, additional effects may appear in the ultimate limit state and serviceability state and the level of safety may be more depend on the quality control. Such effects are, for example, the tilting of the fastener in hole bearing failure or the big distortion of the sheet when the fastener is loaded in tension and the sheet is pulled over the head of the fastener. This is the reason why design procedures for connections in cold-formed elements have been developed which are, in a number of cases, different from the procedures for thicker steel.

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