Joist hangers and framing anchors

STEP lecture C13
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Objectives

To describe the form and load carrying behaviour of common cold-formed steel fasteners for the connection of timber members. To present a design method for joist hangers and framing anchors.

Prerequisites

C5 Nailed joints II

Summary

The application of commonly used cold-formed steel fasteners is shown. The load-carrying behaviour and the capacity of joist hangers under vertical loading is demonstrated, depending on the different components of the connection. A method is given for the design of joist hangers loaded at an angle to the symmetry axis.

Introduction

Joist hangers, framing anchors and other fasteners made from cold-formed steel have widely replaced traditional carpentry joints due to their ease of use and to avoid the need for complex machining of the timber members. Figure 1 shows examples of timber connections using cold-formed steel fasteners. The steel is usually between 1 and 4 *mm* thick and is either hot dip galvanised mild steel or stainless steel.

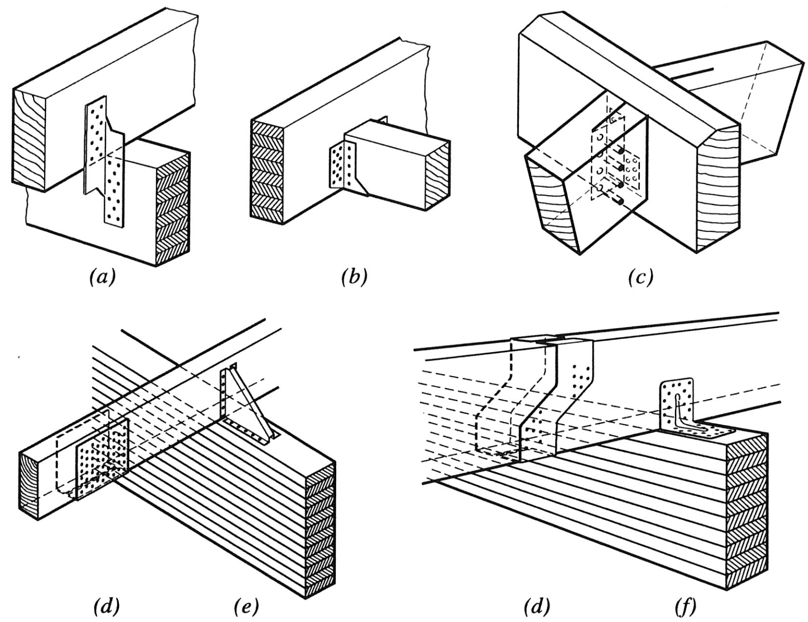

Figure 1 Examples of cold-formed steel fasteners. (a) framing anchor, (b) joist hanger, (c) integral fastener, (d) shear force splices, (e) cleat (f) angle bracket.

The connection between timber and steel is generally nailed using for example annular ringed shank nails without pre-drilling the nail holes in the timber members. The nail holes in the steel fasteners are pre-punched thus allowing simple assembly on the building site.

Load-carrying behaviour

The load-carrying capacity of timber joints with cold-formed steel fasteners not only depends on the nailed steel-to-timber connection but also on the strengths of the timber members and the steel. In particular tensile stresses perpendicular to the grain are likely to cause failure in timber members before the capacity of the nailed connection is reached. Connections prone to such failures include the framing anchor and the joist hanger in Figure 1. Tensile stresses perpendicular to the grain can be taken into account using the design methods given in STEP lecture C2.

In most practical cases rupture of the steel in the net cross-section is prevented by the layout of the fastener. Because the number of pre-punched nail holes limits the force transferred by the nails the steel net cross-section can normally be designed so as not to govern the load-carrying capacity. However, in many connections plastic deformations in the steel fasteners will occur before the maximum load is reached.

In most cold-formed steel fasteners there are at least two steel-to-timber interfaces located in different planes used in the load transfer. The consequent eccentricity causes a combined lateral and axial loading of the nails.

Load carrying capacity of the nailed steel-to-timber connection

The design load-carrying capacity $R_{la,d}$ per nail for single shear joints with a thin steel plate (i.e. for $t \leq 0{,}5\ d$ where t is the thickness) should, according to EC5, be taken as the smaller value found from the following equations:

$$R_{la,d} = \min \begin{cases} 0{,}4\ f_{h,1,d}\ t_1\ d \\ 1{,}1\ \sqrt{2 M_{y,d}\ f_{h,1,d}\ d} \end{cases} \tag{1}$$

For a thick steel plate (i.e. for $t \geq d$) the design value of the load-carrying capacity should be taken as the smaller value found from the following equations:

$$R_{la,d} = \min \begin{cases} 1{,}1\ f_{h,1,d}\ t_1\ d \left[\sqrt{2 + \dfrac{4 M_{y,d}}{f_{h,1,d}\ d\ t_1^2}} - 1 \right] \\ 1{,}5\ \sqrt{2 M_{y,d}\ f_{h,1,d}\ d} \end{cases} \tag{2}$$

If the steel plate thickness lies between 0,5 d and d a linear interpolation is permitted. The difference between the load carrying capacity according to equation (1) and (2), respectively, is caused by the clamping effect of the fastener in the steel plate (see STEP lecture C3). Tests with nailed steel-to-timber joints (Ehlbeck and Görlacher, 1982) have shown, however, that clamping of nails in the steel plate can also occur for steel plates with a thickness of $t = 2{,}0\ mm$ and annular ringed shank nails with a diameter of $d = 4{,}0\ mm$, if the nails are conically shaped close to the nail head (see Figure 2) and are driven in tight fitting holes. In such a case, the load carrying capacities for thick steel plates, according to EC5, are reached.

EC5: Part 1-1: 6.3.1.2a

The characteristic embedding strength $f_{h,1,k}$ depends on the nail diameter d in mm and the characteristic density of the timber ρ_k in kg/m^3 and is for non pre-drilled nail holes:

$$f_{h,k} = 0{,}082\ \rho_k\ d^{-0{,}3}\ N/mm^2 \tag{3}$$

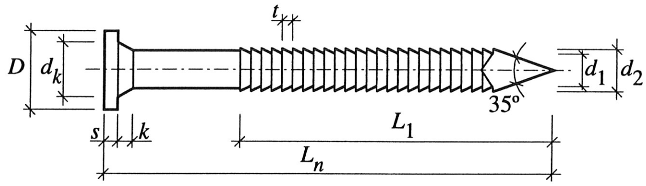

Figure 2 *Annular ringed shank nail for steel-to-timber connections with a cone shaped shank close to the nail head.*

Because of the variable cross section of the nail in the threaded portion and the work hardening during manufacture the characteristic yield moment of the nails has to be determined by bending tests. Calculating the yield moment from the plastic moment of resistance and the tensile strength of the nail wire is not possible for threaded nails. Werner and Siebert (1991) have published test results with annular ringed shank nails produced by four different manufacturers. From the test results, the following characteristic values for the yield moment $M_{y,k}$ of galvanised and stainless steel nails can be derived:

$$M_{y,k} = 6{,}37\ Nm \quad \text{for } d = 4{,}0\ mm \quad \text{and} \tag{4}$$

$$M_{y,k} = 20{,}0\ Nm \quad \text{for } d = 6{,}0\ mm \tag{5}$$

Here, d is the nominal or shank diameter of the nail.

EC5: Part 1-1: 6.3.1.4

The load-carrying capacities of nailed joints according to EC5 have been determined based on minimum nail spacings and distances. Since the nail holes in cold-formed steel fasteners are pre-punched, the nail spacings and some end or edge distances are fixed. In designing joist hangers and framing anchors, care needs to be taken to check the necessary nail spacings and distances. For the spacings a_1 and a_2, it is generally sufficient to check that the area A_n per nail is greater than the value given by the product of the minimum spacings a_1 and a_2 in EC5. It should be noted that for steel-to-timber joints the minimum spacings given for timber-to-timber joints may be multiplied by a factor of 0,7.

$$A_n \geq a_1\ a_2 \tag{6}$$

The design withdrawal capacity $R_{ax,d}$ per fastener for annular ringed shank nails according to EC5 is:

$$R_{ax,d} = f_{1,d}\ d\ l \tag{7}$$

where l is the pointside penetration or the length of the threaded part of the shank, whichever gives the smaller value. The withdrawal capacity according to equation (6) corresponds to a withdrawal of the nail in the member receiving the point. The failure mode related to head pull through does not govern the withdrawal strength in the case of common steel-to-timber joints with steel plate thicknesses of at least 2,0 *mm*. Werner and Siebert (1991) give the following relationship for the parameter $f_{1,k}$ for annular ringed shank nails:

$$f_{1,k} = 65 \cdot 10^{-6}\ \rho^2 \tag{8}$$

where ρ is the timber density in kg/m^3.

Joist hangers

Joist hangers are frequently used as support for sawn timber or glulam beams. Joist hangers are produced in many different shapes and sizes. Figure 3 shows an example of a joist hanger for a timber-to-timber connection.

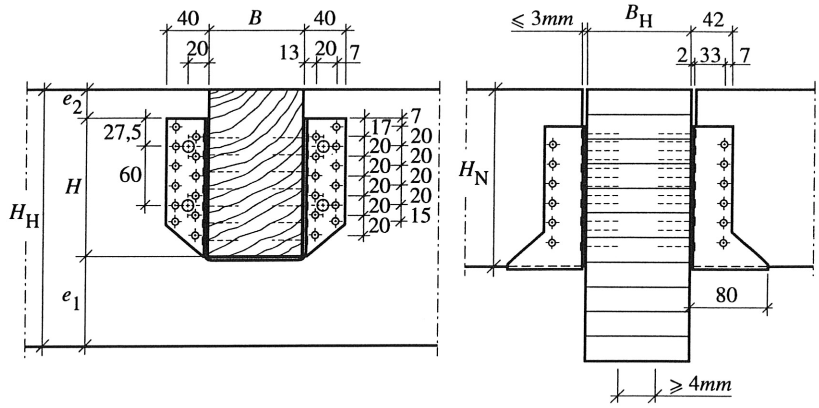

Figure 3 Joist hangers for a secondary beam - main beam connection.

The load acting in the plane of symmetry of the joist hanger connection is transferred from the secondary beam through both the nailed connection and contact with the bottom plate into the joist hanger and then through the nailed connection into the main beam. For joist hangers with only a few nails in the secondary beam, the major part of the load is transferred through contact with the bottom plate. For the design of a joist hanger it can be conservatively assumed that the shear force acts along the line of nails in the secondary beam connection. If the load is mainly transferred through contact, however, the resulting force will normally be closer to the main beam. The connection between the joist hanger and the main beam is consequently loaded by an eccentric force leading to combined lateral and axial loading of these nails.

Joist hangers with straps passing over the main beam often have fewer nails in the main and secondary beam connection. In this type of joist hanger, the loads are mainly transferred by contact into the main beam. Generally, the load carrying capacities of this type of joist hanger have to be derived from tests.

Since the secondary beam nail end distance does not normally satisfy the minimum value specified in EC5, a reduction in load-carrying capacity of the nailed steel-to-timber connection is to be expected. If the nails start to deform, however, the bottom plate of the joist hanger will be loaded by an increasing contact force and take over a larger portion of the load. Riberholt (1975) presented a mechanical model for estimating the contribution of the contact force in the bottom plate. The capacity of the nailed connection and the contact force in the bottom plate can be added since both components have a plastic characteristic.

As an example, the vertical design load-carrying capacity is determined for the joist hanger in Figure 3 with a steel plate thickness of 2 *mm* in service class 1 and for short-term load-duration. The nails used are annular ringed shank nails of the type shown in Figure 2, $d \times l$ = 4,0 x 50 *mm* with a characteristic yield moment $M_{y,k}$ = 6,37 *Nm*. There are 12 nails in the secondary beam and 24 nails in the main beam connection. The timber of both the main and the secondary beam has a characteristic density of 380 kg/m^3. There is a gap of 3 *mm* between the end grain of the secondary beam and the side surface of the main beam.

Nailed secondary beam connection

Since the shear force is assumed to act in the centre of the nailed secondary beam connection, each nail is loaded by the same vertical force.

Nail spacing perpendicular to the grain: $a_2 = 20\ mm > 0{,}7 \cdot 5\ d = 14\ mm$
End distance: $a_{3c} = 32\ mm < 10\ d = 40\ mm$

The load-carrying capacity of the nailed steel-to-timber connection is calculated on the basis of the required minimum nail distances and the assumption that the contribution of the bottom plate may be disregarded. This approach, which has been adopted by German Technical approvals and proved to be conservative by numerous tests with joist hanger connections, is subsequently followed to calculate the load carrying capacity $R_{sb,d}$ of the secondary beam support.

$$R_{sb,d} = n_{sb}\ R_{la,d} \tag{9}$$

Here, n_{sb} is the number of nails in the secondary beam connection and $R_{la,d}$ the design load-carrying capacity per nail. Because of the clamping effect of the nails in the steel plate, equation (2) is used to determine the lateral load carrying capacity per nail. Following the procedure presented in STEP lecture C4 the capacity of the secondary beam connection for the joist hanger in Figure 3 results consequently as

$$R_{sb,d} = 12 \cdot 1{,}22 = 14{,}6\ kN \tag{10}$$

Nailed main beam connection

The eccentricity of the force causes a combined lateral and axial loading of the nails in the main beam connection. The moment is transferred through the axial forces in the nails and the contact force between the end grain surface of the secondary beam and the side surface of the main beam. Tests have shown that after the closing of the gap the bottom edge of the secondary beam bears against the side surface of the main beam. The consequent distribution of the nail withdrawal loads is shown in Figure 4.

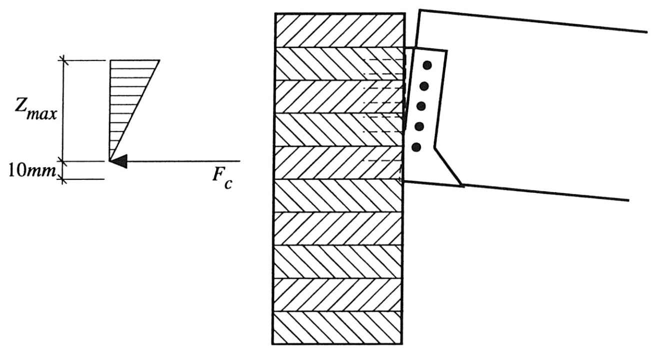

Figure 4 Distribution of withdrawal forces in the nailed main beam connection. F_c is the contact force.

The horizontal contact force F_c acts at the bottom of the secondary beam, the exact position depending on the bearing area. It is assumed that this position, which is also the centre of rotation of the secondary beam end cross-section is located 10 mm above the bottom of the secondary beam. The withdrawal forces of the nails are assumed to increase linearly with increasing distance from the centre of rotation. The maximum withdrawal load for the uppermost nails can then be written as:

$$F_{ax,\max} = \frac{V\ e\ z_{\max}}{\sum_{i=1}^{n_{mb}} z_i^2} \tag{11}$$

where

V	is the force transferred by the joist hanger,
e	is the eccentricity of V with respect to the main beam connection,
n_{mb}	is the number of nails in the main beam connection,
z_i	is the distance of the nail i from the centre of rotation and
$z_{\max}$	is the distance between the uppermost nail in the main beam and the centre of rotation.

EC5: Part 1-1: 6.33

The condition for combined laterally and axially loaded nails is:

$$\left(\frac{F_{ax,d}}{R_{ax,d}}\right)^2 + \left(\frac{F_{la,d}}{R_{la,d}}\right)^2 \le 1 \tag{12}$$

and the lateral load per nail becomes:

$$F_{la,d} = \frac{V}{n_{mb}} \tag{13}$$

Replacing V by $R_{mb,d}$ and substituting F_{ax} and F_{la} in equation (12) by the expressions in equations (11) and (13), respectively, the load carrying capacity of the main beam connection becomes:

$$R_{mb,d} = \frac{1}{\sqrt{\left(\dfrac{e\, z_{\max}}{R_{ax,d} \sum\limits_{i=1}^{n_{mb}} z_i^2}\right)^2 + \left(\dfrac{1}{n_{mb}\, R_{la,d}}\right)^2}} \tag{14}$$

For the joist hanger in Figure 3, the capacity of the main beam connection consequently results as

$$R_{mb,d} = \frac{1}{\sqrt{\left(\dfrac{35 \cdot 133}{907 \cdot 175326}\right)^2 + \left(\dfrac{1}{24 \cdot 1220}\right)^2}} = 22{,}2\ kN \tag{15}$$

The secondary beam connection consequently governs the design. The load carrying capacity of the joist hanger is

$$R_{jh,d} = 14{,}6\ kN \tag{16}$$

Joist hangers loaded at an angle

In most cases, joist hangers used in floors or flat roofs are loaded by a vertical force acting in the direction of the symmetry plane. If joist hangers are used in pitched roofs, however, the load acts at an angle to the principal axes of the joist hanger (see Figure 5).

In this case, the load carrying behaviour differs substantially from the case of uniaxial loading. For joist hangers similar to the one shown in Figure 3 Ehlbeck and Görlacher (1984) have studied the behaviour of joist hangers loaded at an angle to the principal axis. The load carrying capacity of a joist hanger loaded at an angle of 90° is according to Ehlbeck and Görlacher (1984):

$$R_{jh,90} = 0{,}4\ R_{jh,0}\ \frac{h_{jh}}{h_{sb}} \tag{17}$$

where

$R_{jh,90}$	is the load-carrying capacity of the joist hanger loaded at an angle of 90°,
$R_{jh,0}$	is the load-carrying capacity of the joist hanger loaded in the symmetry axis,
h_{sb}	is the depth of the secondary beam with a maximum of 1,5 h_{jh} and
h_{jh}	is the depth of the joist hanger.

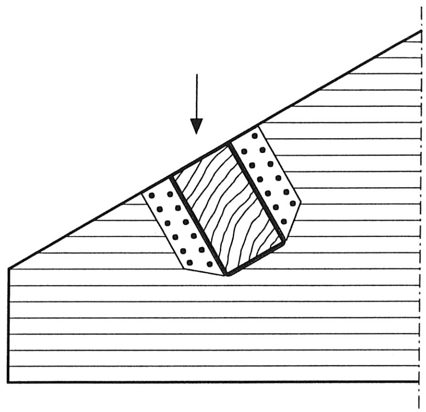

Figure 5 Joist hanger loaded at an angle.

For joist hangers loaded at an angle α between 0° and 90°, the load carrying capacity corresponds to the following interaction equation:

$$\left(\frac{F_{0,d}}{R_{jh,0,d}}\right)^2 + \left(\frac{F_{90,d}}{R_{jh,90,d}}\right)^2 = 1 \tag{18}$$

where $F_{0,d}$ and $F_{90,d}$ are the design values of the load components parallel and perpendicular to the symmetry axis of the joist hanger, respectively.

Framing anchors

Framing anchors are used to connect crossing timber members for example to transfer wind suction forces or as supports for timber beams. In most cases two diagonally positioned framing anchors are used in one connection. There are framing anchors with one and two rows of nail holes, respectively. The type with one row can only transfer tensile forces whereas the type with two parallel rows is able to transfer additional moments.

When designing framing anchors, three different components have to be taken into account:

- load-carrying capacity of the nailed connection. Under the assumption of the load acting in the corner of the framing anchor, the nailed connection is loaded by a force and a moment (see Figure 6),
- load-carrying capacity of the steel net cross-section and
- load-carrying capacity of the timber members. The tensile stresses perpendicular to the grain can be taken into account following the design procedure demonstrated in STEP lecture C2.

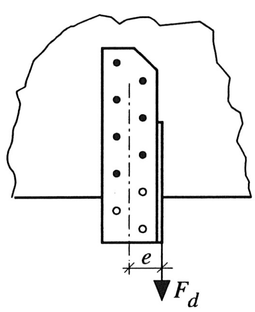

Figure 6 Tensile force in a framing anchor.

For certain types of framing anchors, Römhild (1986) has derived design rules for framing anchors based on an elastic distribution of the lateral nail loads. Based on this approach, the following simplified rules can be used to calculate the load carrying capacity $R_{fa,d}$ of one framing anchor under tensile load:

$$R_{fa,d} = 0,7\ n\ R_{la,d} \quad \text{for anchors with one nail row} \tag{19}$$

$$R_{fa,d} = 0,5\ n\ R_{la,d} \quad \text{for anchors with two nail rows} \tag{20}$$

where n is the number of nails per leg and $R_{la,d}$ the lateral load carrying capacity per nail.

Concluding summary

- Due to easy assembly on site, cold-formed steel fasteners in combination with threaded nails have replaced traditional carpentry joints.

- Nailed connections in cold-formed steel fasteners are mostly loaded eccentrically. In the design of these connections, the steel cross-section as well as the timber members, which are often loaded by tensile stresses perpendicular to the grain, have also to be checked.

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