Frame corners

STEP lecture D8
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Objectives

To describe the types of frame corners commonly used depending on the structural form and the jointing technique. To present design optimisation of the moment-resisting knee joints allowed by EC5.

Summary

Three different types of frame corners are presented:

- tapered members with moment-resisting joints,
- V-shaped columns or knee brace members,
- knee-joints using large finger joints or glued-in bolts.

The applications of these frame corner systems are described and some details are given to ensure the structural performance of the frames. In the second part, a design example of a frame corner presents the possible choices for optimising the dowelled moment-resisting joints commonly used in Europe.

Introduction

Industrial and recreational buildings are often built using a frame with tapered members, or curved frames, as the main structural system. They are simple systems which permit large areas to be covered with spans ranging from 15 to 50 *m* and with spacings of 5 to 10 *m*.

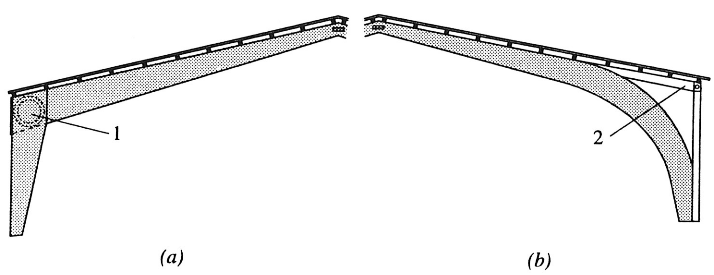

Figure 1 *Frames with tapered members (a) and curved frames (b): (1) knee joints, (2) secondary structural roofing elements.*

The most critical action effects (forces and moments) are found in the frame corner which has to be designed first. This preliminary design defines the largest cross-section for curved frames, or the knee-joint and the largest cross-section for the tapered members. Though curved frames are mechanically more efficient, their use requires:

- special attention to transportation or functional requirements,
- the design of secondary elements (see Figure 1b) depending on the shape of the building and the roofing materials.

Because of these disadvantages, the choice of a frame with tapered members is generally preferred. In this case, the designer is faced with the transfer of the moment and forces between the rafter and the column.

Frame corner systems

Mechanical fastening techniques

The most common frames are built, for spans up to 50 *m*, using tapered members: a single rafter and paired columns (Figure 2a). They are three-hinged to avoid overload due to globally imposed deformations. At the frame corner the internal forces are transferred by using mechanical fasteners placed in a circular or rectangular pattern. To accommodate the fasteners, the mean depth of the members in the area of the joints varies between $L/20$ and $L/30$, where L is the span of the frame. The main disadvantage of this moment-resisting joint is the possibility of splitting induced by moisture content variations or time effects on the joint behaviour. When using dowels, the splits could be reduced by installing dowels reaching ductile failure modes II or III (see STEP lecture C3). In case of toothed-plate connectors, the extent of the splits is limited by staggered location of the connectors in the inner pattern relatively to the outer pattern (Figure 2b). For both types of fastener, reinforcement glued at the end of the members reduces the risk of splitting (Figure 2c).

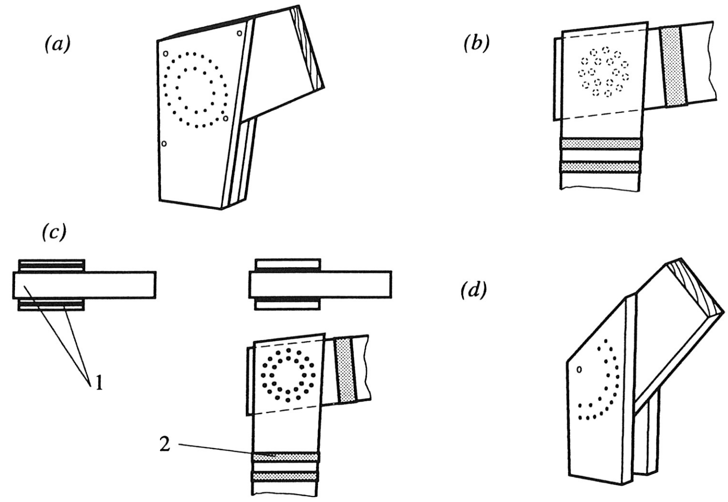

Figure 2 *Frames with mechanically jointed members: (a) typical frame corner, (b) toothed-plate connectors, (c) reinforcement of members: 1 glued-in rod or 2 glued-on plywood, (d) special arrangement for dowelled joint.*

To reduce the stresses near the end-grain of the members, the mechanical behaviour could also be controlled by positioning a stiff rod at the rotation centre. The bending moment is transmitted by a partial pattern of fasteners located along the line of thrust in the area of the joint (Figure 2d). Furthermore, the current trend is to develop knee joints with higher strengths and stiffnesses and with ductile behaviour. In European countries, this has led to studies involving side reinforcement of the members in the joint area using glued plywood or densified veneer wood (Leijten et al., 1994) or fibre glass. These reinforcements prevent overloading of the end corner of the members and ensure plastic behaviour at failure.

Another way, used in Japan and Australia, is to design single in-plane members (rafter and column) and to install internal steel plates with dowel fasteners or external plywood gussets (Figure 3). The first arrangement is a good solution for appearance and fire resistance. In both cases, the equilibrium of forces and moment is achieved in the centre section of the plate and the designer has to pay attention to the internal strength of the plate.

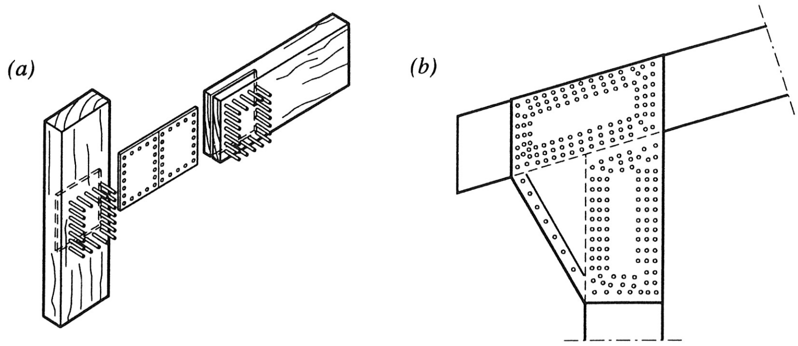

Figure 3 *Frames with mechanically jointed in-plane members: (a) joint with internal steel plate, (b) nailed plywood gusset*

V-shaped column

Another concept is to change the loading path using a V-shaped column fixed to the rafter (Figure 4). Depending on the stability criteria and horizontal action effects, this type of frame could be two-hinged with a continuous pitched and curved beam with spans up to 30 *m*. For greater spans, they should be three-hinged. The depths of the beam vary between *L*/30 and *L*/40 for h_1, and *L*/40 to *L*/60 for h_2. To transfer the bending moment, the tension and compression members should make an angle α within the range 10 to 20°.

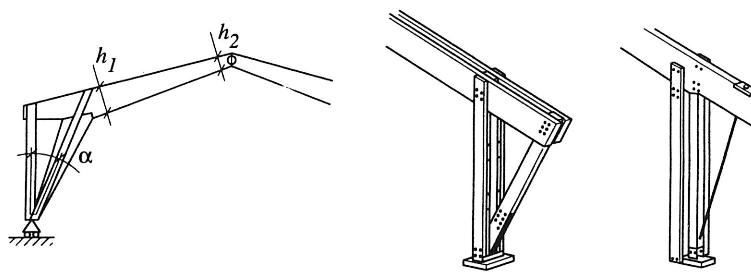

Figure 4 *V-shaped frame geometry.*

With the internal timber member in compression, either a vertical internal timber member or an external steel rod in tension may be used. For such a frame, the design must investigate all combined action effects to take into account possible reversal of loading in the column members.

Glued jointing techniques

This technique is carried out using large finger joints or glued-in bolts (Figure 5). Mainly developed in Scandinavian countries, these glued joints lead to high local force transfer and very stiff connections. The main restriction in use results from the brittle behaviour of these joints. So, their production and installation require extended quality control and must fulfil specific requirements as specified for large finger joints in prEN387, "Glued laminated timber - Production requirements for large finger joints".

Members connected by large finger joints is the more common glued joint for frames. The joint profile is cut along the depth of the member. In general the knee joint is produced by installing a corner block and cutting two large finger joints. This enables the angle between the forces and the grain direction to be reduced and hence the joint strength is increased. Furthermore, special attention should also be paid to reversal of loading as the strength of large finger joints is greater for negative moment (M_{neg})

than for positive values (M_{pos}). The absolute value of the ratio M_{pos}/ M_{neg} varies from 0,1 to 0,2 for a roof slope of 0 to 30° (Reyer et al., 1991, Heimeshoff, 1976).

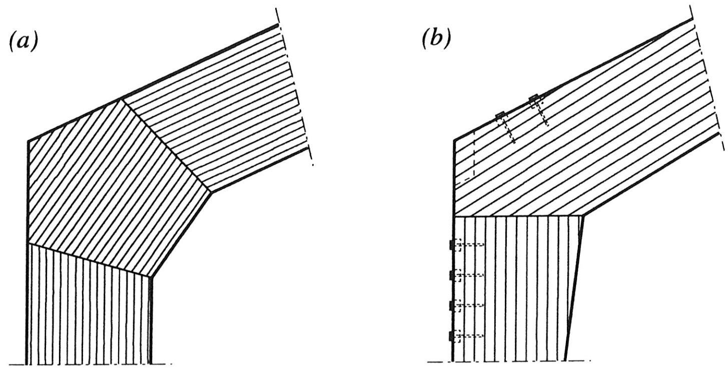

Figure 5 Frame corners using large finger joint (a), or glued-in bolts (b).

As on-site installation is not allowed for large finger joints, the use of such non-demountable joints is also limited by transportation conditions. To overcome this disadvantage and to limit the risk associated with a single component brittle joint, another solution is to used glued-in bolts (see STEP Lecture C14). Figure 6 shows an installation with inclined glued-in bolts to reduce the influence of possible splitting on the joint strength (Turkowskij, 1991). In this example, forces and moment are transferred by separate compression and tension components and low load reversal is permitted.

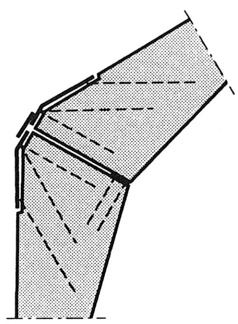

Figure 6 Frame corner with inclined glued-in bolts.

Design of frame corners

Especially for dowelled moment-resisting joints, EC5 rules and linked standards allow new possibilities for frame optimisation. Aimed at increasing the use of timber, the first stage is to design the most efficient joints (mechanically and economically). This section presents a design example showing the possible choice for designers and the timber industries.

A three-hinged frame (Figure 7) is designed with glued-laminated tapered members. Related to the short-term load duration class, the critical load combination gives the forces at the knee-joint in the column and the rafter:

column: $V_{c,d} = 80 \cdot 10^3\ N \qquad N_{c,d} = 120 \cdot 10^3\ N$

rafter: $V_{r,d} = 90 \cdot 10^3\ N \qquad N_{r,d} = 112 \cdot 10^3\ N$

and $M_{c,d} = 260 \cdot 10^3\ Nm$

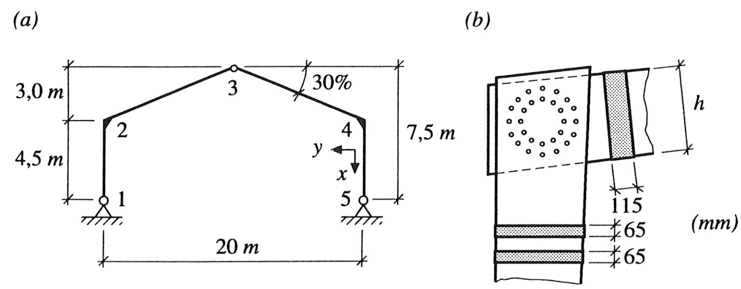

Figure 7 Geometry of the frame (a) and layout of the frame corner (b).

Current design (frame A)

Based on a preliminary design, the geometry and the properties of the selected materials are:

Glued-laminated members:
strength class GL24 (prEN 1194, annex A)

ρ_k	$= 364\ kg/m^3$	$f_{v,k}$	$= 2,52\ N/mm^2$	k_{mod}	$= 0,9$
t_1	$= 65\ mm$	t_2	$= 115\ mm$	h	$= 1150\ mm$

Dowels:
steel grade Fe240 $\quad f_{u,k} = 240\ N/mm^2$
$\quad d = 24\ mm.$

A circular pattern is chosen and the radii are:

$$r_1 = \frac{h}{2} - 4\,d = 460\ mm \quad , \quad r_2 = r_1 - 5\,d = 340\ mm$$

With a minimum spacing of 6 d between dowels, the number of dowels per circle is:

$$n_1 \leq \frac{2\,\pi\,r_1}{6\ d} = 20 \qquad n_2 \leq \frac{2\,\pi\,r_2}{6\ d} = 14$$

The design joint forces induced by the forces and the bending moment are defined in Table 1 (see STEP Lecture C16).

		Column	Rafter
F_M	$(10^3\ N)$	20,4	20,4
F_V	$(10^3\ N)$	2,35	2,65
F_N	$(10^3\ N)$	3,53	3,29
F_d	$(10^3\ N)$	23,1	23,3
$F_{v,d}$	$(10^3\ N)$	157	152

Table 1 Design values of the forces on the most critical fasteners and the timber.

The mechanical properties of the fasteners have the following values:

embedding strength: $$f_{h,0,d} = \frac{0,9}{1,3}\ 0,082\ (1\ -\ 0,01 \cdot 24)\ 364\ =\ 15,7\ N/mm^2$$

yield moment: $$M_{y,d} = \frac{0,8 \cdot 240 \cdot 24^3}{1,1 \cdot 6} = 402 \cdot 10^3\ Nmm$$

The coefficient k_{90} is equal to 1,71. The calculation of the design load-carrying capacity of the critical dowels (see STEP lectures C6 and C16) is given in Table 2.

		Column	Rafter
α_1	(°)	81,2	81,9
α_2	(°)	7,9	8,6
$f_{h,1,d}$	(N/mm^2)	9,27	15,5
$f_{h,2,d}$	(N/mm^2)	15,5	9,26
β		1,67	0,6
R_d	($10^3\,N$)	14,5 21,4 11,9 16,5	24,1 12,8 13,0 16,5

Table 2 Design resistance per shear plane for the critical dowels.

For the chosen patterns, the load-carrying condition is checked on the column:

$$R_{j,d} = 2 \cdot 11{,}9 \cdot 10^3 = 23{,}7 \cdot 10^3\ N > F_{d,c} = 23{,}1 \cdot 10^3\ N$$

and on the rafter:

$$R_{j,d} = 2 \cdot 12{,}8 \cdot 10^3 = 25{,}6 \cdot 10^3\ N > F_{d,c} = 23{,}3 \cdot 10^3\ N$$

In the joint area, the strength of the timber is verified for the calculated force $F_{V,d}$:

$$\tau_d = \frac{3\ F_{v,d}}{2\ b\ h} = \frac{3 \cdot 152 \cdot 10^3}{2 \cdot 115\ 1150} = 1{,}72\ N/mm^2 < f_{V,d} = \frac{0{,}9}{1{,}3}\ 2{,}52 = 1{,}74\ N/mm^2$$

At the serviceability limit states, the rotational rigidity of the joint is:

$$K_{ser,r,d} = K_{ser}\ (n_1\ r_1^2 + n_2\ r_2^2) = 9{,}75 \cdot 10^{10}\ Nmm/rd$$

Design with improved steel grade (frame B)

Choosing dowels of smaller diameter and greater tensile strength, EC5 allows the plastic behaviour of the moment-resisting joint to be improved. In this example, the design option is to select dowels of steel grade Fe430 ($f_{u,k}$ = 430 N/mm^2) with a diameter of 24 *mm*.

An optimum design is obtained for h = 1080 *mm* and 45 (30 + 15) dowels installed on two circular patterns with the radii r_1 = 470 *mm* and r_2 = 390 *mm*. Table 3 gives the results of the design calculations.

F_M	($10^3\,N$)	13,7	$f_{h,0,d}$	(N/mm^2)	17,3
F_V	($10^3\,N$)	1,78	$f_{h,1,d}$	(N/mm^2)	11,0
F_N	($10^3\,N$)	2,67	$f_{h,2,d}$	(N/mm^2)	17,2
F_d	($10^3\,N$)	15,7	$R_{j,d}$	($10^3\,N$)	7,9
$F_{V,d}$	($10^3\,N$)	140	τ_d	(N/mm^2)	1,69

Table 3 Results of the design calculations.

For the calculation of the deflection in service, the design has to take into account the joint rigidity:

$$K_{ser,r,d} = K_{ser}\ (n_1\ r_1^2 + n_2\ r_2^2) = 9{,}90 \cdot 10^{10}\ Nmm/rd$$

Design with improved glued laminated timber and steel grade (frame C)
The other way to optimise the timber structures is to select glued laminated timber of a greater strength class. It should be mentioned that this choice implies greater control requirements during the fabrication process. For this design, the selected components are:

Glued-laminated members:
strength class GL30 (prEN1194, annex A)
ρ_k = 407 kg/m^3 $\quad f_{v,d}$ = 2,18 N/mm^2

Dowels:
steel grade Fe430 $\quad f_{u,k}$ = 430 N/mm^2

The possible designs are:
C1 h = 1080 mm and 41 (30 + 11) dowels d = 16 mm,
C2 h = 980 mm and 70 (38 + 32) dowels d = 12 mm.

At the serviceability limit states, the rotational rigidity of the joint is:

C1 $K_{ser,r,d} = 109 \cdot 10^9\ Nmm/rd$ $\qquad$ C2 $K_{ser,r,d} = 118 \cdot 10^9\ Nmm/rd$

Design summary

	Glued laminated timber		dowels	
Design	class	h (mm)	steel grade	number and diameter
A	GL24	1150	Fe240	34 d 24 mm
B	GL24	1080	Fe430	45 d 16 mm
C1	GL30	1080	Fe430	41 d 16 mm
C2	GL30	980	Fe430	70 d 12 mm

Table 4 Design possibilities of the moment-resisting joint.

In comparison with design A, the best cost-efficiency could be reached with the design B or C2 depending on the cost of each operation during the fabrication and erection processes.

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