Strength and stability of structures

Timber structures

Ympäristöministeriö
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Foreword

The Ministry of the Environment publishes the recommendations for strength and stability related to the design of timber structures in the National Building Code of Finland. The instruction contains a compilation of all the national annexes concerning the design of timber structures.

The beginning of each National Annex presents those clauses in the standard where national choice is permitted, and where such a choice has been made.

Helsinki, 20 December 2016

Head of the Buildings and Construction unit
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Contents

1. Scope

These instructions provide additional information when applying the Ministry of Environment Decree on load-bearing structures in the design and execution of timber structures. Where applicable, the instructions also apply to timber structures used in composite constructions, such as timber and concrete structures or timber and steel structures. A solution pursuant to these instructions is considered to meet the requirements set for load-bearing structures.

These instructions are applied when timber structures are designed pursuant to standards SFS-EN 1995 and their Finnish national annexes, and executed pursuant to standard SFS-EN 5978.

Furthermore, harmonised product standards concerning timber products may contain supplementary rules concerning the design of timber structures.

2. Design of structures

2.1 Execution documents

Standard SFS-EN 5978 provides instructions on the preparation of the execution documents and the execution specification for timber structures.

Usually, the execution documents include, at a minimum, the following:
a) construction drawings
b) requirements pursuant to standard SFS-EN 5978, such as the execution classes and tolerance classes
c) other documents to be adhered to or references to other documents.

2.2 Contents of the structural designs

Usually, the structural designs for timber structures present, at a minimum, the following to the extent applicable to the design task:
a) consequences class
b) the structure's service class and the planned service life of the structure
c) the R/E/I/M fire resistance class and fire behaviour class for the structural components
d) the adopted characteristic loads and category of use
e) complete information on the dimensions and location of the structures
f) execution class

g) tolerance class

h) identifying information for the materials and supplies

i) any allowances for moisture expansion possibly required at joints and seams

j) stiffening of structures during installation at the end

k) information concerning the surface and protection coatings and proofing.

The following are also presented for factory-made construction components (included in manufacturing or installation drawings):

l) the information required for the assessment of the qualification and design of the building product

m) the CE labelling method adopted for the construction component (M1, M2, M3a or M3b)

n) the weight and centroid location for the timber element

o) the minimum support surfaces

p) lifting points

q) handling, support and lifting instructions if necessary.

2.3 Execution classes

The requirements set for the implementation of timber structures are divided into three execution classes according to the difficulty of the structure. The execution classes are presented in standard SFS 5978.

The execution class is selected on the basis of standard SFS-EN 1990 and the consequences classes (CC1, CC2 and CC3) and risk factors related to the implementation. Consequences class CC2 structures are in execution class TL2, at a minimum, and consequences class CC3 structures are in execution class TL3. Consequences class CC2 timber structures in buildings with over 3 storeys or over 14 m in height are in execution class TL3. Those consequences class CC2 structures whose use or execution involves special risk factors are in execution class TL3.

A section or detail of a timber structure may be assigned an execution class that differs from the rest of the structure.

2.4 Durability and design working life

When examining the durability of timber structures, account is taken of the structural protection of the timber structures, the biological durability of the timber and timber products, and the corrosion resistance of the metal connectors and connecting parts. Structural protection is the goal regardless of whether timber is used as such or with chemical or physical modifications. Any possible effects of

the modifications on the corrosion of the metal connectors and connecting parts are taken into account.

Standard SFS 5978 presents instructions regarding the durability of timber structures and corrosion protection for fasteners that supplement standard SFS-EN 1995-1-1.

3. Execution

3.1   Execution planning

The work plans for the execution of timber structures are drawn up on the basis of the execution documents in adherence with standard SFS-EN 5978.

Usually, the work plans for the execution of timber structures present, at a minimum, the following to the extent applicable to the design task:
-    the required execution drawings
-    the work phase plans pursuant to standard SFS 5978 required by the execution documents
-    quality documents pursuant to standard SFS 5978.

The harmful saturation, drying and exposure to continuous sunlight during transport, storage at the worksite and installation is prevented by means of sufficient protection.

If joints in timber structures whose resistance is utilised at the ultimate limit state are manufactured at the worksite by means of adhesives or nailed plates, the same instructions are followed as in factory manufacturing, while taking into account the additional requirements caused by manufacturing at the worksite.

3.2   Building products used

The characteristics of the building products, materials and supplies used in timber structures are demonstrated by means of the CE label if they are covered by the scope of the harmonised product standard or if the manufacturer has acquired the European Technical Approval/Assessment for its product. Otherwise, they are demonstrated according to the Act on the Type Approval of Certain Construction Products (954/2012).

The characteristics of the following products are central in terms of the reliability of the timber structures:
-    solid timber, laminated timber, LVL and cross laminated timber (CLT)

- structural wood-based panels
- fasteners
- mechanically connected or glued joists, columns and lattices
- timber elements
- sheeting fasteners
- building boards used for wind stiffening
- composite timber and concrete slabs
- fireproofing products

The strength and stiffness properties of unsorted solid timber in pole form, round timber, planed round log and solid wood may be considered to correspond to strength class C24 of similar solid timber, assuming that the properties of the timber correspond to timber grown in Finland. This timber does not include gluing or glued end joints.

The strength class of coniferous sawn timber does not need to be determined when the party engaging in a building project acquires the logs and saws them, or has them sawn, for their own use for a future one-family house or agricultural building. The strength class of such solid timber can be assumed to be C24, assuming that the timber corresponds to timber grown in Finland.

Sawn solid timber means that the timber has been sawn from flawless logs and that the only sorting performed on the timber after the sawing has been based on dimensions and the removal of pieces containing defects, such as wane or rot. Solid timber that has not been strength classified cannot be used as raw material for timber with glued end joints, glued laminated timber or joint fastener structures.

For factory-manufactured or partially factory-manufactured timber elements and structural members that are not covered by obligatory CE labelling, manufacturing quality control that aims at ensuring conformity is performed pursuant to the applicable standards and/or acceptance criteria. Factory manufacturing and quality control shall, at a minimum, follow the regulations and instructions concerning the worksite implementation and control of a similar structure.

Gluing that is only utilised at the serviceability limit state may be performed without external quality control, but even in this case, glue joints must have a strength and durability that keeps the seam intact throughout the planned service life.

4. Execution supervision and the conformity of structures

4.1 Execution supervision

The inspections related to the supervision of the execution of timber structures are drawn up within the scope required by the execution documents in adherence with standard SFS 5978.

During the execution of the structures, the responsible work supervisor or a separately appointed specialist work supervisor will supervise that the plans and instructions concerning the manufacture of timber structures and the installation of the timber elements are followed and that the appropriate documents are prepared for the work.

If it is observed during the execution that a structure or detail does not meet the requirements laid down in the execution documents, the occurrence locations and causes of the deviations are analysed. This is done to determine whether the deviation can be approved without a repair. If necessary, calculations are used to demonstrate that the confidence interval required by standard SFS-EN 1995 and its national annexes is achieved. If it cannot be demonstrated that the deviation is acceptable without a repair, the repair will be carried out to the necessary extent.
The deviation and corrective action will be recorded in the quality control archive.

The quality control material is documented and compiled into a single entity.

4.2 Conformity of structures

When applying these instructions, the suitability appraisal for structures is based on the timber structures being designed appropriately pursuant to standards SFS-EN 1995 and their national annexes, and on the timber structures being executed and inspected pursuant to the execution documents.

5. References

If the version of a reference has not been specified, the latest edition of the reference (with amendments) is applied.

SFS-EN 1990   Eurocode. Basis of structural design

SFS-EN 1995-1-1   Eurocode 5: Design of timber structures. Part 1-1: General rules and rules for buildings

SFS-EN 1995-1-2   Eurocode 5: Design of timber structures. Part 1-2: General. Structural fire design

SFS-EN 5978   Execution of timber structures. Rules for load-bearing structural members in buildings

6. National annexes to Eurocodes SFS-EN 1995

National Annex to standard SFS-EN 1995-1-1 Part 1-1: General rules and rules for buildings

As regards standard SFS-EN 1995-1-1, the recommended values set forth in standard SFS-EN 1995-1-1 and all the annexes to standard SFS-EN 1995-1-1 are followed unless otherwise stated in this National Annex.

The Non-Contradictory Complementary Information (NCCI) is presented in italics.

National choice is permitted in the following clauses of standard SFS-EN 1995-1-1:
- 2.3.1.2(2)P
- 2.3.1.3(1)P
- 2.4.1(1)P
- 6.1.7(2)
- 6.4.3(8)
- 7.2(2)
- 7.3.3(2)
- 8.3.1.2(4), Note 2
- 8.3.1.2(7)
- 9.2.4.1(7)
- 9.2.5.3(1)
- 10.9.2(3)
- 10.9.2(4).

A national choice has been made in the clauses marked ●.

Load-duration classes

2.3.1.2(2)P
Examples of load-duration assignment are given in Table 1.

Table 1. Examples of load-duration assignment

Load-duration class	Loading
Permanent	Self-weight Machinery, equipment and lightweight partition walls fixed permanently to the structure Earth pressure
Long-term	Storage loads (category E), water tank load
Medium-term	Snow Uniformly distributed imposed loads on floors and balconies in categories A–D Imposed loads on garages and trafficable areas (categories F and G) Actions due to moisture variation
Short-term	Imposed loads on stairs Concentrated imposed load (Q_k) Horizontal loads on partition walls and parapets Maintenance load or load caused by persons on a roof (category H) Vehicle loads in category E Actions due to transport vehicles Installation loads
Instantaneous	Wind Accidental action

Service classes

2.3.1.3(1)P

The following sets out additional information on the assignment of structures to service classes given in (2)P, (3)P and (4)P:

Timber structures in heated rooms or in corresponding moisture conditions belong to service class 1. Generally, any structures in thermal insulation and joists with their tension side within thermal insulation may also be included in service class 1.

Service class 2 includes dry timber structures outdoors. Structures should be in a covered and ventilated space and well protected underneath and on the sides from getting wet. For instance, timber structures in a ground floor and cold attic space are usually included in this service class.

Service class 3 includes timber structures exposed to weather, located in a damp space outside or subject to the immediate effect of water.

In addition to the average moisture content, attention should be paid to moisture variation in the assignment of structures to service classes. The effect of moisture variation

on a timber structure may be greater than the effect of a high average moisture level. Particular attention should be paid to the risk of timber splitting in service class 1.

Design value of material property

2.4.1(1)P

The recommended values given in Table 2.3 are used for the partial factors of the strength and stiffness properties of materials. However, in punched metal plate joints, a value of 1.1 is used as the partial factor when the sheet is made of steel.

The resistances of steel members in a connection are verified according to SFS-EN 1993 with the partial safety factors of material properties given in the National Annex to SFS-EN 1993.

Ultimate limit states

Chapter 6

The design of holes in bellows can be performed according to the NCCI 1 document that follows this National Annex.

Columns subjected to either compression or combined compression and bending

6.3.2

In normal support cases, the values given in Table 2 are used as the lateral buckling length of a compressed rod.

Table 2. Lateral buckling lengths L_c for a compressed rod when the rod length is L.

Support method	Lateral buckling length L_c
The rod is fixed at one end and has a joint at the other end, structure with no sway	*0.85 L*
The rod has joints at both ends	*1.0 L*
The rod has transverse supports in the lateral buckling direction at intervals a	*1.0 a*
The rod has rigid fastening at one end and is free at the other end	*2.5 L*

Beams subjected to either bending or combined bending and compression

6.3.3
The dimension $l_{ef} = a + 2h$ may be used as the effective lateral torsional buckling length of a joist that is transversely supported at the compressed edge at intervals a, when h is the height of the joist and the load acts along the compressed edge of the joist, and the value $l_{ef} = a - 0.5h$ can be used when the load acts along the tensioned edge. If the compressed edge of the joist is only loaded by point loads located at the lateral torsional buckling supports, the effective lateral torsional buckling length of $l_{ef} = a$ may be adopted.

Shear

6.1.7(2)
The following values are used for the factor k_{cr}:

$k_{cr} = 0.67$ for solid timber in heated rooms or in corresponding moisture conditions

$k_{cr} = 1.0$ for laminated timber

$k_{cr} = 1.0$ for solid timber in moisture conditions that permanently correspond to service class 2 or 3

$k_{cr} = 1.0$ for timber products pursuant to standards SFS-EN 13986 and SFS-EN 14374

Double tapered, curved and pitched cambered joists

6.4.3(8)
The tensile stresses perpendicular to grain in double tapered, curved and pitched cambered joists may be evaluated using expression (6.55), if the timber has a surface treatment preventing the moisture transfer. Otherwise, expression (6.54) is applied.

Joint slip

7.1
In the serviceability limit state bending and vibration design, the utilisation degree of gluing performed at the site without external quality control is limited to 50% of the full joint effect.

Limiting values for deflections of beams

7.2(2)
When the deflection of a structure or horizontal deflection of a building is harmful, the serviceability limits of deformations for the characteristic combinations of actions are

according to Table 3 unless due to the type of structure, purpose of use or nature of activity other values can be considered more appropriate. If the wind load is not the leading variable action, it may be disregarded in the loading combinations of serviceability limit states.

Table 3. Limiting values for deflections. Deflection of cantilevers may be double. *L* is the span width and *H* is the height of the considered point in the building.

Structure	w_{inst}[1]	$w_{net,fin}$	w_{fin}[2]
Main girders	$L/400$	$L/300$	$L/200$
Purlins and other secondary girders	-	$L/200$[3]	$L/150$
Horizontal displacement of a building	-	$H/300$	-

[1] Only applies to floors

[2] Applies to pre-cambered structures and structures with a curve/fold between the supporting points

[3] When calculating the deflection of the floor plate, the load is a short-term point load $Q_k = 2$ kN and the self-weight of the plate

Residential floors

7.3.3(2)
The NCCI 2 document that follows this National Annex is used in the vibration design of floors; it completely replaces clause 7.3.3 of the standard and meets all the criteria presented therein.

Nailed timber to timber connections

8.3.1.2(4), Note 2
Instructions in clause 8.3.1.2(4) are followed when determining the shear rupture resistance of a joint that is nailed at the end of a timber piece in the grain direction.

8.3.1.2(7)
The instructions in clause 8.3.1.2(7) are not applied to nail joints.

Axially loaded nails

8.3.2
The design of longitudinally loaded nail joints takes into account the reduction of anchorage strength caused by timber drying. The nail anchorage strength that has been experimentally defined at a relative air humidity of RH65 is reduced during the design by applying a factor of 0.4 for nails with smooth shafts and 0.7 for other nails, if the joint

is used in a heated indoor environment. Nails with smooth shafts must not be used to bear the long-term longitudinal nail loads that occur in ceiling fastening, for example.

Trusses

9.2.1

The thickness of the timber used for punched metal joint structures is at least 42 mm. If the span of the punched metal joint structure is more than 18 m, the thickness of the timber is at least 45 mm or, alternatively, punched metal structures joined together at the factory may be used; for these, the effective slenderness of the combined com-pressed rods corresponds to a unified rod that is at least 48 mm in thickness.
When the punched metal joint structure has a span of over 5 m, the width of the flanges and other outer rods is at least 90 mm and the width of the inner rods is at least 68 mm. The width of the transversely supported inner rods in the punched metal joint structure (rods supported against buckling) is at least 120 mm. The maximum number of trans-versely supported inner rods in the punched metal joint structure is at most 1 + L/5, when L is the structure's overall length in metres. At this point, rods that will be used for the attachment of permanent wall or roof structures are not counted as transversely sup-ported inner rods.

General

9.2.4.1(7)

The simplified analysis for shear walls is carried out pursuant to method A in clause 9.2.4.2.

Bracing of beam or truss systems

9.2.5.3(1)

The values given in Table 4 are used for the modification factors of transverse stiffening.

Table 4. Modification factor values.

Modification factor	Value
k_s	$2 + 2\cos\left(\dfrac{180°}{m}\right)^{*)}$
$k_{f,1}$	50
$k_{f,2}$	80
$k_{f,3}$	50

*$^{*)}m$ is the number of fields transversely supported at intervals $a \geq 2$ (see Figure 9.9 of standard SFS-EN 1995-1-1).

Erection

10.9.2(3)

The highest allowable lack of rod straightness at field interval $a_{bow,perm}$ = 15 mm. The largest allowed lateral lack of straightness across the entire length of the flange $a_{bow,perm}$ = min(L/300; 50 mm), when L is the length of the flange.

10.9.2(4)

Largest permitted deviation from the vertical position $a_{dev,perm}$ = min(10 mm + H/200; 25 mm), when H is the height of the lattice [mm] at the point of local application.

Annex A
Block shear and plug shear failure at multiple dowel-type steel- totimber connections

Annex A is applied in Finland to tensioned rod end joints.

With LVL, expression (A.1) in standard SFS-EN 1995-1-1 receives the format

$$F_{bs,Rk} = max \begin{cases} 1{,}25 A_{net,t} f_{t,0,k} \\ 0{,}7 A_{net,v} f_{v,0,flat,k} \end{cases} \qquad (1.1)$$

where $f_{v,0,flat,k}$ is the characteristic value of LVL's plane shear strength in the grain direction of the face veneers.

Block shear should also be checked in connection with rod end joints between tensioned timber parts that have two or multiple shears. For joints between timber parts, expression (A.3) in standard SFS-EN 1995-1-1 can be reduced to

$A_{net,v} = L_{net,v} t_1$ $\qquad (1.2)$

NCCI 1 for standard SFS-EN 1995-1-1: Design of joist holes

This instruction concerns laminated timber and LVL joists with holes when the holes are round or rectangular and curved at the corners with a curvature radius of at least r = 15 mm.

When hole diameter d > 50 mm, the dimensions presented in Figure 1 must meet the following conditions:

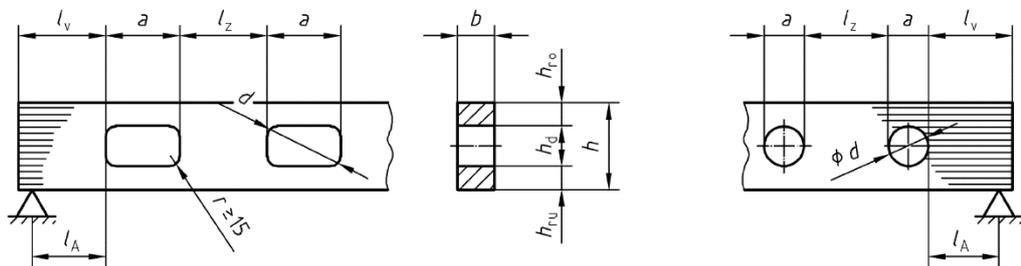| $l_v \geq h$	$l_z \geq 1.5h$, however ≥ 300 mm	$l_A \geq h/2$	$h_{ro} \geq 0.35h$, $h_{ru} \geq 0.35h$	$a \leq 0.4h$	$h_d \leq 0.15h$, round, $\varnothing_d \leq 0.3h$

Figure 1. Symbols for a holed joist.

The following condition must be met:

$$\sigma_{t,90,d} = \frac{F_{t,90,d}}{0,5 \cdot b \cdot k_{t,90} \cdot l_{t,90}} \leq f_{t,90,d} \qquad (1.1)$$

where

b	is the width of the joist
$k_{t,90} = min\{1; (450/h)^{0,5}\}$	where h is the height of the joist in millimetres
$f_{t,90,d}$	is the design value for the tree's transverse tensile strength
$l_{t,90} = \begin{cases} 0,5 \cdot (h_d + h) \\ 0,35 \cdot d + 0,5 \cdot h \end{cases}$	rectangular holes round holes.

Transverse tensile force $F_{t,90,d}$ depends on the joist shear force V_d acting at the hole and the bending moment M_d as follows:

$$F_{t,90,d} = \frac{V_d \cdot h_d}{4 \cdot h} \cdot \left(3 - \frac{h_d^2}{h^2} \right) + 0,008 \cdot \frac{M_d}{h_r} \qquad (1.2)$$

$$h_{\mathrm{r}} = \begin{cases} \min(h_{\mathrm{ro}}; \ h_{\mathrm{ru}}) & \textit{rectangular holes} \\ \min(h_{\mathrm{ro}} + 0{,}15d; \ h_{\mathrm{ru}} + 0{,}15d) & \textit{round holes} \end{cases}$$

h, h_{ro}, h_{ru} and d   *are defined in Figure 1*

hd   *is the height of the opening for rectangular holes and h_d = 0,7d for round holes.*

Furthermore, the shear resistance, bending resistance and tensile/compression resistance of the joist are checked at the hole for a cross-section where the hole part has been deducted. In this analysis, only the surface of the hole is deducted, while the other peak tension analyses are included in the instruction presented above.

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NCCI 2 for standard SFS-EN 1995-1-1: Floor vibration caused by walking

Vibration caused by walking is considered in the serviceability limit state design, while observing the intended use and occupancy of the building or room.

The design may adopt the following floor vibration design method that completely replaces clause 7.3.3 of standard SFS-EN 1995-1-1 and meets all the criteria presented therein.

The following criteria are applied to tenements with permanent residence and to offices:

Special analyses are required if the fundamental frequency of the floor structure in a residential or commercial tenement is below 9 Hz ($f_1 < 9$ Hz).

When the floor frequency of a residential or commercial tenement has a fundamental frequency $f_1 > 9$ Hz, the meeting of the following condition is checked:

$\delta \leq 0.50$ mm   (1.1)

where δ is the largest momentary deflection caused by a static point load of 1 kN at the floor joist. In small rooms, the permitted deflection of 0.5 mm may be increased by the factor k in Figure 1.
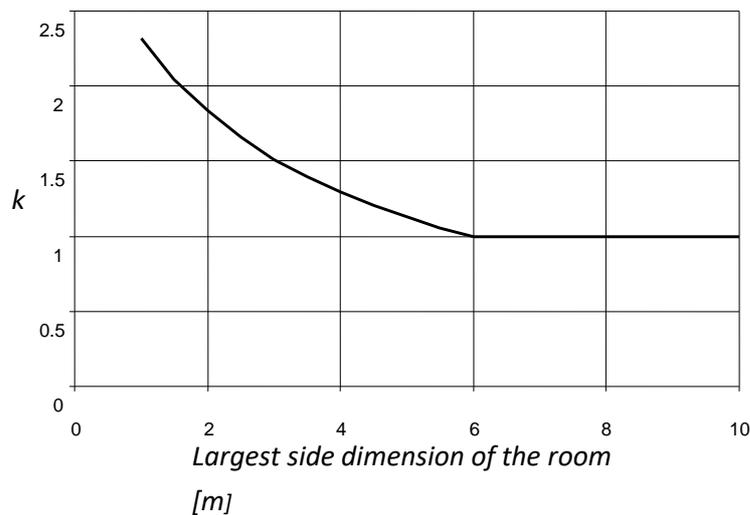

Figure 1. *The room-size-dependent increase factor k for the bending limitation*

The fundamental frequency for a one-way spanning floor structure can be calculated with the expression

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$$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}} \qquad\qquad (1.2)$$

The fundamental frequency for a two-way spanning floor structure can be calculated with the expression

$$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_l}{m}} \cdot \sqrt{1 + \left[2 \cdot \left(\frac{l}{b}\right)^2 + \left(\frac{l}{b}\right)^4\right] \cdot \frac{(EI)_b}{(EI)_l}} \qquad (1.3)$$

where

l	is the span of the floor structure [m]
b	is the width of the floor structure [m]
$(EI)_l$	is the floor's bending stiffness in the span direction by unit of width [Nm^2/m]
$(EI)_b$	is the floor's bending stiffness in the transverse direction by unit of width [Nm^2/m]
m	is the combined mass of the floor's self-weight per unit of area and a share of the imposed load of 30 kg/m2 [kg/m2].

In the case of a one-way spanning floor structure, the deflection caused by a point load (F = 1 kN) located at the floor joist can be calculated with the expression

$$\delta = min \begin{cases} \dfrac{Fl^2}{42 \cdot k_\delta \cdot (EI)_l} \\[2mm] \dfrac{Fl^3}{48 \cdot s \cdot (EI)_l} \end{cases} \qquad\qquad (1.4)$$

where

s	is the distance between floor joists [m]

$$k_\delta = \sqrt[4]{\frac{(EI)_b}{(EI)_l}} \quad \text{limitation } k_\delta < b/l \qquad\qquad (1.5)$$

Expression (1.4) may also be adopted for a floor supported on four sides. In this case, factor k_δ pursuant to expression (1.5) does not need to be limited with < b/l.

This instruction can also be applied as such for continuous floor joists or slabs with two or more holes. In this case, however, the floor structure must not be continuous between different tenements.

National Annex to standard SFS-EN 1995-1-2 Part 1-2: General. Structural fire design

As regards standard SFS-EN 1995-1-2, the recommended values set forth in standard SFS-EN 1995-1-2 and all the annexes to standard SFS-EN 1995-1-2 are followed unless otherwise stated in this National Annex.

The Non-Contradictory Complementary Information (NCCI) is presented in italics.

National choice is permitted in the following clauses of standard SFS-EN 1995-1-2:
- 2.1.3(2)
- 2.3(1)P
- 2.3(2)P
- 2.4.2(3), Note 2
- 4.2.1(1).

A national choice has been made in the clauses marked ●.

Parametric fire exposure

2.1.3(2)
No values are given for the average temperature rise $\Delta\theta_1$ and for the maximum temperature rise $\Delta\theta_2$ during the cooling phase of fire.

The requirement for the separation function is only based on a standard fire and on temperature limits set by it.

The fire safety requirement is also deemed to be satisfied if the building is designed and executed based on design fire scenarios which cover the situations likely to occur in the said building. The satisfaction of the requirement is attested case-by-case, taking into consideration the properties and use of the building.

Member analysis

2.4.2(3), Note 1
The determining of the reduction factor uses partial factors from standard SFS-EN 1990 and the Ministry of Environment Decree 3/16 concerning its application, in which case Figure 2.1 in standard SFS-EN 1995-1-2 will change as presented in Figure 1.
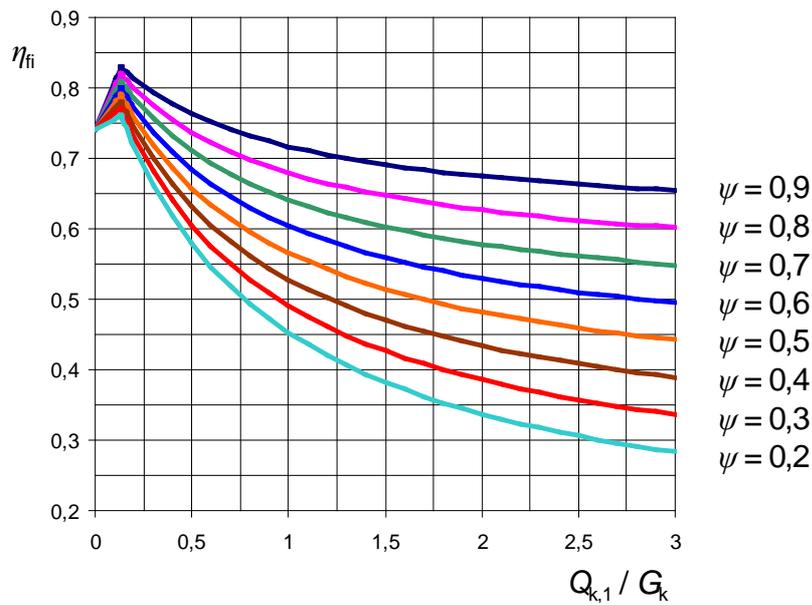
Figure 1. *The variation of the reduction factor η_{fi} as a function of the load ratio of the nominal values of dominant variable action and permanent action $Q_{k,1}\,/\,G_k$ according to the load combination rules presented in the Ministry of Environment Decree 3/16 concerning the application of standard SFS-EN 1990.*

2.4.2(3), Note 2
No approximate values are presented for the reduction factor

General

4.2.1(1)
The method in clause 4.2.2 is used for the definition of the section properties.

General
E.1(3)
Design values of type A can be used for gypsum plasterboard of type R.

The test method for walls is presented in standards EN 1364-1 (non-load-bearing) and EN 1365-1 (load-bearing) and the test method for floors in standard EN 1365-2.

Annex C

Load-bearing floor joists and wall studs in assemblies whose cavities are completely filled with insulation

Annex C is not used.

Instead of Annex C, the document NCCI 1 that follows this National Annex is used.

Annex D

Charring of members in wall and floor assemblies with void cavities

Annex D is not used.

Instead of Annex D, the document NCCI 2 that follows this National Annex may be used.

Annex E

Analysis of the separating function of wall and floor assemblies

Annex E may only be used for the analysis of wall structures.

Annex F

Guidance for users of this Eurocode Part

Reduced cross-section method is chosen as design procedure for mechanical resistance in the flow chart F.1 of Informative Annex F.

NCCI 1 for standard SFS-EN 1995-1-2: Load-bearing floor joists and wall studs in assemblies whose cavities are completely filled with insulation

1. General

This annex discusses the load-bearing capacity of wall and floor structures with timber frames. Sheathing on the side of the fire will protect the timber frame parts (studs and joists) against a standard fire for a maximum of 60 minutes. The following conditions are met:

- *the cavities are completely filled with mineral wool (rock fibre or glass fibre)*
- *transverse binding prevents the buckling of the studs at the wall level and the lateral torsional buckling of the floor joists; the binding is created by sheathing or transverse support binds on the opposite side*
- *in floors, the sheets can also be attached to steel profiles with a maximum height of 25 mm and which are perpendicular to the direction of the joists*
- *compartmentation is demonstrated pursuant to clause 5.3 of standard SFS-EN 1995-1-2.*

The method may be used even when the cavities are not completely filled if the thickness of the insulation is at least 100 mm and the density is at least 30 kg/m3. The insulation shall be at the level of the narrow side of the member exposed to fire, so that it protects the wide sides of the member from charring.

2. Residual cross-section

2.1 Charring rates

The nominal residual cross-section is specified according to Figure 1, where the nominal charring depth is taken from expression (3.2) in standard SFS-EN 1995-1-2 and the nominal charring rate is specified according to expression (1.1) or (1.2).
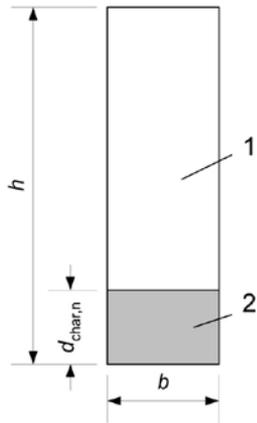
Key:

1   Nominal residual cross-section

2   Nominal charcoal layer

Figure 1. *Nominal residual cross-section in a load-bearing timber wall stud or block joist located inside a cavity and protected by insulation*

The nominal charring rate for a timber joist or stud that is protected by sheathing on the side of the fire is calculated as follows:

$$_n \, \kappa_s \, k_2 \, k_{n\,0} \qquad\qquad\qquad \text{when } t_{ch} \le t \le t_f \qquad\qquad\qquad (1.1)$$

$$_n \, \kappa_s \, k_3 \, k_{n\,0} \qquad\qquad\qquad \text{when } t \ge t_f \qquad\qquad\qquad (1.2)$$

where:

$k_n = 1.5$

β_v	*is the design value for the nominal charring rate*
k_s	*is the cross-section factor pursuant to Table 1*
k_2	*is the insulation factor pursuant to Table 2 or 3*
k_3	*is the post-protection factor pursuant to Table 2 or 3*
k_n	*is the factor used to convert an irregular residual cross-section to a nominal rectangular cross-section*
β_0	*is the design value for the charring rate of one-dimensional charring according to Table 3.1 of clause 3.4.2 in standard SFS-EN 1995-1-2*
t	*is the time variable for fire exposure*
t_{ch}	*is the starting time for the charring of a load-bearing timber structural member pursuant to clause 2.2*
t_f	*is the rupture point of the cladding pursuant to clause 2.3.*

The cross-section factor is taken from Table 1.

Table 1. Cross-section factors for load-bearing timber joists and studs of different widths

b mm	k_s
38	1.4
45	1.3
60	1.1
≥ 90	1.0

Intermediate values are interpolated.

The insulation factor k_2 values in Table 2 are used for floors and values in Table 3 are used for walls. The values are not dependent on joint configurations.

Table 2. Charring starting time t_{ch} and cladding failure time t_f and factors k_2 and k_3 for floor structures

Cladding	t_{ch}	k_2	t_f	k_3 [1] / k_3 [2]
	min		min	
A	10	-	10	3.0/4.0
2 x A [3]	30	-	30	3.0/4.0
A + F [3,4]	40	0.85	45	3.8/5.0
F [4]	15	0.85	30	3.8/5.0
2 x F [4]	60	0.85	> 60	-
Pl + F [4,5]	40	0.85	45	4.0
Pl + A [3,5]	30	-	30	3.0

[1] If the insulation is supported in a manner where there is no charring on the vertical sides of the girders
[2] If the insulation is supported by steel profiles, wood siding or chicken wire (vertical sides not completely uncharred)
[3] Sheet A 13 mm gypsum plasterboard
[4] Sheet F 15 mm gypsum plasterboard
[5] Sheet Pl 12 mm gypsum plasterboard or other wood-based panel. If the thickness d of plywood or wood-based panel is more than 12 mm, the values of t_{ch} and t_f in the table are increased by Δt, when $Δt = (d – 12) / \beta_0$.

Table 3. Charring starting time t_{ch} and cladding failure time t_f and factors k_2 and k_3 for wall structures

Cladding	t_{ch}	k_2	t_f	k_3
	min		min	
A	15	-	15	1.5
2 x A [1]	40	-	40	1.0
A + F [1,2]	55	0.85	>60	-
F [4]	20	0.85	50	3.8
2 x F [4]	65	0.85	> 60	-
Pl + F [2,3]	55	0.85	>60	-
Pl + A [1,3]	40	-	40	1.0

[1] *Sheet A 13 mm gypsum plasterboard*
[2] *Sheet F 15 mm gypsum plasterboard*
[3] *Sheet Pl 12 mm gypsum plasterboard or other wood-based panel. If the thickness d of plywood or wood-based panel is more than 12 mm, the values of t_{ch} and t_f in the table are increased by Δt, when $\Delta t = (d - 12) / \beta_0$.*

The post-protection factor k_3 values in Table 2 are used for floors and values in Table 3 are used for walls. The values given for floor structures depend on how the insulation is supported.

If cavity insulation is done with glass wool, fire design of wall and floor structures is performed according to method given in Annex D to standard SFS-EN 1995-1-2 (Charring of members in wall and floor assemblies with void cavities).

2.2 Start of charring

When the fire protection cladding is made of wood-based panels, the value used for the start of charring of timber structural members t_{ch} is:

$$t_{ch} = t_f \tag{1.3}$$

where failure time t_f is calculated pursuant to clause 2.3.

Where the fire protective claddings are made of gypsum plasterboard of type A, R or F or of the combination of these boards and wood-based panels (gypsum plasterboard outermost), the time of start of charring t_{ch} in Table 2 shall be used for floors and the values in Table 3 for walls.

2.3 Time of cladding failure

The following is used for the time of failure of wood panel cladding:

$$t_f = \frac{h_p}{\beta_0} - 4 \qquad\qquad (1.4)$$

where:

t_f	*is the time of failure in minutes*
h_p	*is the sheet thickness in millimetres*
β_0	*is the design value for the charring rate of one-dimensional charring in a standard fire (mm/min).*

The time of failure for cladding made of type F gypsum plasterboard is determined:

- *based on the thermal disintegration of the cladding*
- *based on pull-out failure due to the insufficient penetration of connectors into un-burned wood.*

Where the fire protective claddings are made of gypsum plasterboard of type A, R or F or of the combination of these boards and wood-based panels (gypsum plasterboard outermost), the time cladding failure t_f in Table 2 shall be used for floors and the values in Table 3 for walls.

The sheet failure time t_f caused by pull-out failure can be calculated with the expression:

$$t_f = t_{ch} + \frac{l_f - l_{a,\min} - h_p}{k_s k_2 k_n \beta_0} \qquad\qquad (1.5)$$

where:

t_{ch}	*is the starting time of charring*
l_f	*is the length of the fastener*
$l_{a,min}$	*is the minimum penetration of the fastener into unburned timber*
h_p	*is the total cladding thickness*
k_s	*is the cross-section factor pursuant to clause 2.1*
k_2	*is the insulation factor pursuant to Table 2 or 3*
k_n	*is the factor pursuant to clause 2.1 used to convert an irregular residual cross-section to a nominal rectangular cross-section*

β_0 is the design value for the charring rate of one-dimensional charring in a standard fire according to Table 3.1 of clause 3.4.2 in standard SFS-EN 1995-1-2

A value of 10 mm is adopted for the minimum penetration $l_{a,min}$ into unburned wood.

When the cladding is attached to steel profiles as in Figure 2, the failure time of the steel profiles can be calculated according to expression (1.5), where h_p is replaced by the steel profile thickness t_s.
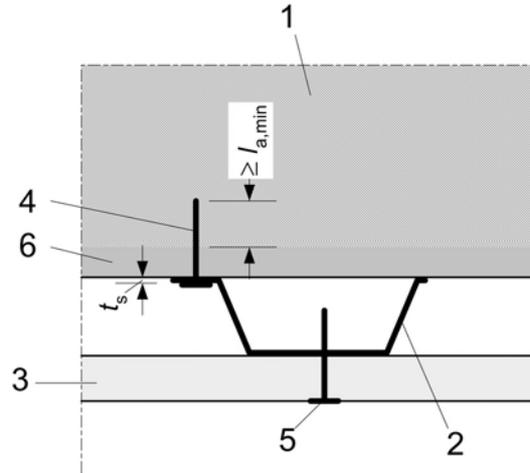
Key:
1 Timber structural member
2 Steel profile
3 Sheet
4 Fastener connecting the steel profile to the timber structural member
5 Fastener connecting the sheet to the steel profile
6 Charcoal layer

Figure 2. A drawing that describes the fastening of the sheets to the roof

When steel profiles are used for keeping the insulation inside the cavity in place after the failure of the sheeting, the time of failure that corresponds to pull-out failure can be calculated with the expression:

$$t_{sf} = t_f + \frac{l_f - l_{a,min} - k_s k_2 k_n \beta_0 (t_f - t_{ch}) - t_s}{k_s k_3 k_n \beta_0} \qquad (1.6)$$

where:

t_{sf} is the time of failure of the steel profiles
t_s is the thickness of the steel profile
k_3 is the post-protection factor

other symbols are explained in connection with expression (1.5).

When fire resistance is ≤ 60 min, the resistance and stiffness of the steel profiles does not need to be demonstrated.

3. Reduction of strength and stiffness parameters

The modification factor that is used to take into account the effect of fire on the strength of load-bearing timber structural members is calculated with the expression:
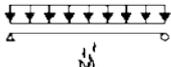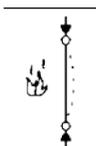
$$k_{\text{mod},fm,fi} = a_0 - a_1 \frac{d_{char,n}}{h}$$

(1.7)

a_0, a_1 *are values pursuant to Tables 4 and 5*

$d_{char,n}$ *is the nominal charring depth pursuant to expression (3.2) of standard SFS-EN 1995-1-2, where βn is calculated with expressions (1.1) and (1.2)*
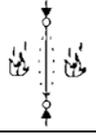
h *is the height of the cross-section of the joist or stud.*

Table 4. The values for a_0 and a_1 required for reducing the strength value of the joist or studs when only one side of the floor or wall is exposed to fire

Case			h mm	a_0	a_1
1	Bending strength with exposed side in tension		95	0.60	0.46
			145	0.68	0.49
			195	0.73	0.51
			220	0.76	0.51
			300	0.84	0.51
			400	0.94	0.51
			500	1.00	0.51
2	Bending strength with exposed side in compression		95	0.46	0.37
			145	0.55	0.40
			195	0.65	0.48
			220	0.67	0.47
			300	0.73	0.47
			400	0.81	0.47
			500	0.89	0.47
3	Compression resistance		95	0.46	0.37
			145	0.55	0.40
			195	0.65	0.48
			220	0.67	0.47

The values for the other heights a_0 and a_1 in Table 4 are interpolated.

Table 5. The values for a_0 and a_1 required for reducing the strength value of the stud when both sides of the wall are exposed to fire

Case			h mm	a_0	a_1
1	Compression resistance		145	0.39	1.62

The modification factor for the modulus of elasticity is calculated with the expression:

$$k_{\mathrm{mod},E,fi} = b_0 - b_1 \frac{d_{char,n}}{h} \tag{1.8}$$

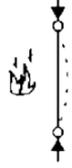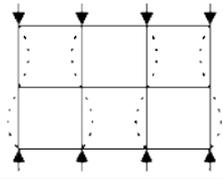*where:*

b_0, b_1 are the values pursuant to Tables 6 and 7

$d_{char,n}$ is the nominal charring depth pursuant to expression (3.2) of standard SFS-EN 1995-1-2, where β_n is calculated with expressions (1.1) and (1.2)

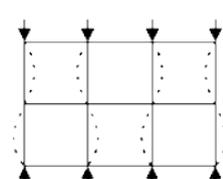h is the height of the joist.

Table 6. The values for a_0 and a_1 required for reducing the modulus of elasticity of the stud when only one side of the wall is exposed to fire

Case			h mm	b_0	b_1
1	Perpendicular out-of-plane buckling		95	0.50	0.79
			145	0.60	0.84
			195	0.68	0.77
2	In-plane buckling		95	0.54	0.49
			145	0.66	0.55
			195	0.73	0.63
NOTE: In the drawing for case 2, the studs are laterally bound by lacing.					

The values for the other heights b_0 and b_1 in Table 6 are interpolated.

Table 7. The values for a_0 and a_1 required for reducing the modulus of elasticity of the columns when both sides of the wall are exposed to fire
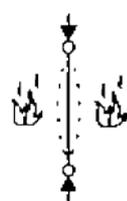
Case			h mm	b_0	b_1
1	Perpendicular out-of-plane buckling		145	0.37	1.87
2	In-plane buckling		145	0.44	2.18
NOTE: In the drawing for case 2, the studs are laterally bound by lacing.					

NCCI 2 for standard SFS-EN 1995-1-2: Charring of members in wall and floor assemblies with void cavities

General

The rules in this annex apply to standard fire.

Clauses 3.4.3.2(1), (2), (4) and (5) from standard SFS-EN 1995-1-2 are used.

Charring rate

For fire protective claddings made of gypsum plasterboard or of the combination of wood-based panel and gypsum plasterboard, the insulation factor values k_2 in Table 2 of the NCCI 1 document following the National Annex to standard SFS-EN 1995-1-2 are used for floors and the values in Table 1 for walls.

Starting time of charring

When the fire protection cladding is made of wood-based panels or as wood panelling, the following value is used for the starting time of the charring of a timber joist or stub:

$$t_{ch} = t_f \hspace{5cm} (1.1)$$

where t_f is determined pursuant to expression (1.2).

For fire protective claddings made of gypsum plasterboard or of the combination of wood-based panel and gypsum plasterboard, the charring starting time value t_{ch} in Table 2 of the NCCI 1 document following the National Annex to standard SFS-EN 1995-1-2 are used for floors and the values in Table 1 for walls. The same time of start of charring is used for the narrow and wide sides of a member.
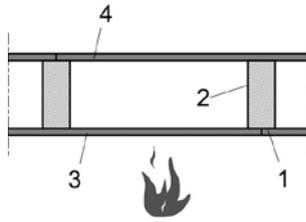
Key:

1   Side of joist or stud exposed to fire
2   Wide side of joist or stud facing the cavity
3   Fire protection cladding on the side exposed to fire
4   Fire protection cladding on the side opposed to the fire exposure

Figure 1. *Definition of narrow and wide side of timber joist or stud*

Time of cladding failure

The value used for the time of failure t_f for fire protection cladding that is attached to wood joists or studs and implemented as wood panelling or wood-based panels:

$$t_f = \frac{h_p}{\beta_0} - 4 \qquad\qquad (1.2)$$

where:

t_f *is the time of failure in minutes*

h_p *is the sheet thickness in millimetres*

β_0 *is the design value for the charring rate of one-dimensional charring [mm/min].*

For fire protective claddings made of gypsum plasterboard or of the combination of wood-based panel and gypsum plasterboard, the time of failure values t_f in Table 2 of the NCCI 1 document following the National Annex to standard SFS-EN 1995-1-2 are used for floors and the values in Table 1 for walls.

Table 1. Charring starting time t_{ch} and cladding failure time t_f and factors k_2 and k_3 for wall structures.

Cladding	t_{ch}	k_2	t_f	k_3
	min		min	
A	15	-	15	2.0
2 x A [1]	40	-	40	2.0
A + F [1,2]	55	0.85	77	2.0
Pl + F [2,3]	55	0.85	77	2.0
Pl + A [1,3]	40	-	40	2.0

[1] *Sheet A 13 mm gypsum plasterboard*
[2] *Sheet F 15 mm gypsum plasterboard*
[3] *Sheet Pl 12 mm gypsum plasterboard or other wood-based panel. If the thickness d of plywood or wood-based panel is more than 12 mm, the values of t_{ch} and t_f in the table are increased by Δt, when $\Delta t = (d - 12) / \beta_0$.*

Resistance is determined by means of the effective cross-section method, clause 4.2.2 of standard SFS-EN 1995-1-2, using clause 4.2.2(3) of standard SFS-EN 1995-1-2 to determine factor k_0.