NATIONAL BUILDING CODE OF FINLAND

Strength and stability of structures

Steel structures

Ympäristöministeriö
Miljöministeriet
Ministry of the Environment

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Foreword

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

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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 steel structures. Where applicable, the instructions also apply to steel parts used in composite steel and concrete structures. A solution pursuant to these instructions is considered to meet the requirements set for load-bearing structures.

These instructions are applied when steel structures are designed pursuant to standards SFS-EN 1993 and their National Annexes, and executed pursuant to standard SFS-EN 1090-2.

2. Design of structures

2.1 Execution documents for structures

Usually, the execution documents include, at a minimum, the following:

a) construction drawings
b) requirements pursuant to standard SFS-EN 1090-2, such as the execution classes, tolerance classes, prefabrication grades and the information required in Annex A of standard SFS-EN 1090-2
c) requirements pursuant to standard SFS-EN 1090-4, such as the execution classes, tolerance classes, prefabrication grades and the information required in Annex F of standard SFS-EN 1090-4
d) other followed documents or references to other documents
e) if necessary, steelwork not covered by SFS-EN 1090-2 and SFS-EN 1090-4 (such as the fire protection work for the steel structure).

2.2 Contents of the structural designs

Usually, the structural designs for steel structures present, at a minimum, the following to the extent applicable to the design task:

a) consequences class
b) corrosivity category and the design working life of the structure
c) the R/E/I/M fire resistance 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) the allowed geometrical deviations pursuant to standards SFS-EN 1090-2 and SFS-EN 1090-4 and special project-specific tolerances (e.g. geometry of structural parts, location of joints and connections, nominal sizes and locations of holes if deviations that differ from the tolerance classes in standards SFS-EN 1090-2 and SFS-EN 1090-4 are required)

h) identifying information for the constituent products
i) technical specification required for designing the fire protection (e.g. critical temperature during standard fire) or plans concerning performance-based fire safety design fire design or other fire design plans
j) fire protection method
k) weld quality level
l) utilisation ratio for welds and, if necessary, welds intended for special inspection
m) the strength class of the filler metal when the steel grade is at least S500
n) the effective a-dimensions of welds required for the calculations and butt weld thicknesses regardless of the welding process and, if necessary, the effective lengths of tack welds ℓ_{eff} and the effective lengths of butt welds
o) surface preparation requirements for the corrosion protection method
p) whether methods apart from tightening are used in order to secure nuts against coming loose
q) special requirements concerning manufacture, such as the execution of holes, weld grinding, allowed hardness values, allowed bending radii for cold bending and holes required by galvanising, unless these are presented separately elsewhere in the execution documents. Special requirements refer to matters that affect the resistance of the structure or that are otherwise required on the basis of planning
r) corrosion allowance in unpainted or unprotected steel.

The following are also presented for structural components (included in manufacturing or installation drawings):
s) the information required for the assessment of the qualification and design of the fabricated product
t) the method used in CE marking for the structural component (M1, M2, M3a or M3b)
u) the weight and centroid location for the structural component
v) lifting points
w) handling, support and lifting instructions if necessary.

2.3 Execution classes

The execution class is selected based on standard SFS-EN 1993-1-1 and its National Annex.

The requirements set for the implementation of steel structures are divided into four execution classes according to the difficulty of the structure. The requirements for each execution class are presented in standard SFS-EN 1090-2.

2.4 Durability and design working life

In order to achieve the design working life, the corrosivity category are defined according to the environmental conditions. The corrosivity category is used to determine the requirements such as the

steel grade to be used, the method of protection and the inspection and maintenance activities required by it.
The corrosion allowance required for unpainted and unprotected steel structures is determined by means of calculation by applying the corrosion rates given in standard ISO 9224:1992 in environmental corrosivity categories C1–C5, unless otherwise stated in standards SFS-EN 1993 and their National Annexes.

3. Execution

3.1 Execution planning

The work plans for the execution of steel structures are drawn up on the basis of the execution documents in adherence with standard SFS-EN 1090-2 and SFS-EN 1090-4.

Usually, the work plans for the execution of steel structures present, at a minimum, the following to the extent applicable to the design task:
- the required execution drawings
- installation plan pursuant to standard SFS-EN 1090-2, SFS-EN 1090-4, or both
- a fire protection plan that normally presents the following, at a minimum:
  - the product name and possible product approval identifier of the fire protection agent; in the case of fire protection paint, more specific information concerning the fire protection paint system
  - the component-specific design value for the fireproofing (e.g. the thickness of the fire protection plate or insulation or the dry film thickness of the fire protection paint)
  - instructions for the periodic condition inspection of the fire protection, which are appended to the operating and maintenance instructions of the building
  - quality documents pursuant to standard SFS-EN 1090-2, SFS-EN 1090-4, or both.

3.2 Construction products

The characteristics of the constituents products used in steel structures are demonstrated by means of the CE mark 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 steel structures:
- steel profiles and plates
- sheetings

- screws and bolt assemblies
- welding consumables
- fire protection products
- structural components.

When using constituent products presented in the reference standards presented in standards SFS-EN 1090-2 and SFS-EN 1090-4 that have no harmonised product standard, their material properties are usually demonstrated with inspection documents pursuant to the requirements of standards SFS-EN 1090-2 and SFS-EN 1090-4.

3.3 Structural components

If a harmonised product standard does not exist for a fabricated steel product, and the manufacturer cannot present a European Technical Approval/Assessment (ETA), and the conformity of the building product has not been demonstrated with a voluntary product approval procedure pursuant to Act 954/2012, the conformity of the structural component is demonstrated on a per-site basis by using the quality documents pursuant to the requirements of standards SFS-EN 1090-2 and SFS-EN 1090-4, or both.

3.4 Fire protection for steel structures

The installation of fire protection for steel structures is carried out while taking special care and in accordance with the possible product approval for the fire protection product, general instructions concerning fire protection products and any other product-specific instructions from the manufacturer.

Unless otherwise stated in the voluntary product approval concerning the fire protection of steel structures with fire protection paint, the following instructions are to be followed during painting:
- The performer of the fire protection maintains a record of the work phases for fire protection painting; it is used to record the information on the painting conditions, the drying interval between the paint layers and the film thickness measurements.
- The steel surface that is primed or directly painted with fire protection paint is cleaned, at a minimum, to the Sa 2½ degree of prefabrication in standard SFS-EN ISO 8501-1. The cleaning of the galvanised surface is performed according to the product-specific instructions for the fireproof paint that can be used on galvanised surfaces.
- The suitability of the fire protection paint for the location and the compatibility of the fire protection paint, primer and top coating are verified. The nominal thickness of the film thickness of the primer shall be at least 40 μm.
- The film thicknesses of the protected steel structures shall be measured at the painting place. A dry film thickness measurement is performed on the primer, and it is used to determine the average primer film dry thickness. The thickness of the wet fire protection paint film is regularly monitored by means of a comb gauge. If necessary, the dry film thickness may also be measured

after the application of each layer. When measuring the final film thickness for fire protection paint, the thickness of the primer is taken into account on the basis of the average measured dry film thickness.

- The dry film thickness measurements are performed according to standard SFS-EN ISO 2808. At least 10 % of the painted structures need to be measured. The selected areas must be as representative of the different surfaces of the different surfaces of the painted structure as possible. The measurement points for a typical steel profile are selected from surfaces pursuant to Figure 1.
    - Open profiles: 2 measurements / m from inner and outer surfaces
    - Hollow sections: square and rectangular: 2 measurements / m from all outer surfaces
    - Hollow sections: circular: 8 measurements / m from equal distances around the cross-section
    - In the case of short or unusual profiles the abovementioned amounts shall be applied.
- If the average film thickness within the measurement area falls below the required film thickness, a necessary number of additional measurements is performed in order to determine the area with insufficient film and the need for additional painting. The average film thickness of one side of the painted profile shall not be less than 80 % of the required film thickness. The maximum value of the film thickness must not exceed the value reported by the paint manufacturer.
- Structures with fire protection painting are protected against weather and moisture exposure and mechanical damage during transport, storage and the entire duration of the construction.

Figure 1. The dry film thickness measurement for fire protection paint is performed at the locations indicated in the figure

4. Execution supervision and the conformity of structures

4.1 Execution supervision

The inspections related to the supervision of the execution of steel structures are drawn up within the scope required by the execution documents in adherence with standard SFS-EN 1090-2 and SFS-EN 1090-4.

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 in-situ structural component 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 standards SFS-EN 1993 and their 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. The quality control documentation of steel structures consists of the quality control documentation compiled at the factory and during the execution at the worksite.

4.1.1 Fire protection supervision

The following information related to the fire protected structure is included in the structural designs and operating and maintenance instructions of the building:

- The product name, type and manufacturer
- Product approval and it’s validity
- The fire protection contractor and inspector
- The structure’s R fire class used in the design (R15 – R120)
- Possible repairs and their inspector
- The installation time for the fire protection
- The inspection time

Unless more detailed instructions are presented, the fireproofing is inspected visually at least once every three years. Damage to the fireproofing will be repaired according to instructions from the fire protection material manufacturer.

The documentation concerning the fire protected structures will be compiled for inclusion in the operating and maintenance instructions of the building.

4.2 Conformity of structures

When applying these instructions, the suitability appraisal for structures is based on the steel structures being designed appropriately pursuant to standards SFS-EN 1993 and their National Annexes, and on the steel 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 1090-2	Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures
SFS-EN 1090-4	Execution of steel structures and aluminium structures – Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications
SFS-EN 1990	Eurocode. Basis of structural design
SFS-EN 1993-1-1	Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings
SFS-EN 1993-1-2	Eurocode 3: Design of steel structures. Part 1-2: General rules. Structural fire design
SFS-EN 1993-1-3	Eurocode 3: Design of steel structures. Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting
SFS-EN 1993-1-4	Eurocode 3: Design of steel structures. Part 1-4: General rules. Supplementary rules for stainless steel
SFS-EN 1993-1-5	Eurocode 3: Design of steel structures. Part 1-5: Plated structural elements
SFS-EN 1993-1-6	Eurocode 3: Part 1-6: Strength and stability of shell structures

SFS-EN 1993-1-7	Eurocode 3: Design of steel structures. Part 1-7: Plated structures subject to out of plane loading
SFS-EN 1993-1-8	Eurocode 3: Design of steel structures. Part 1-8: Design of joints
SFS-EN 1993-1-9	Eurocode 3: Design of steel structures. Part 1-9: Fatigue of steel structures
SFS-EN 1993-1-10	Eurocode 3: Design of steel structures. Part 1-10: Material toughness and through-thickness properties
SFS-EN 1993-1-11	Eurocode 3: Design of steel structures. Part 1-11: Design of structures with tension components
SFS-EN 1993-1-12	Eurocode 3: Design of steel structures. Part 1-12: Extension of SFS-EN 1993 up to steel grades S700
SFS-EN 1993-3-1	Eurocode 3: Design of steel structures. Part 3-2: Towers, masts and chimneys. Towers and masts
SFS-EN 1993-3-2	Eurocode 3: Design of steel structures. Part 3-2: Towers, masts and chimneys. Chimneys
SFS-EN 1993-4-1	Eurocode 3: Design of steel structures. Part 4-1: Silos
SFS-EN 1993-4-2	Eurocode 3: Design of steel structures. Part 4-2: Tanks
SFS-EN 1993-5	Eurocode 3: Design of steel structures. Part 5: Piling
SFS-EN 1993-6	Eurocode 3: Design of steel structures. Part 6: Crane supporting structures
SFS-EN ISO 2808	Paints and varnishes. Determining film thickness
SFS-EN ISO 8501-1	Preparation of steel substrates before application of paints and related products. Visual assessment of surface cleanliness. Part 1: Rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings
ISO 9224:1992	Corrosion of metals and alloys --_Corrosivity of atmospheres - - Guiding values for the corrosivity categories

6. National annexes to standards SFS-EN 1993

National Annex to standard SFS-EN 1993-1-1 Part 1-1: General rules and rules for buildings

As regards standard SFS-EN 1993-1-1, the recommended values set forth in standard SFS-EN 1993-1-1 and all the annexes to standard SFS-EN 1993-1-1 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 1993-1-1:

- ● 2.3.1(1), Note 1
- ● 3.1(2)
- ● 3.2.1(1)
- ● 3.2.2(1)
- ● 3.2.3(1)
- ● 3.2.3(3)B
- – 3.2.4(1)B, Note 3B
- – 5.2.1(3)
- ● 5.2.2(8)
- – 5.3.2(3)
- ● 5.3.2(11), Note 2
- – 5.3.4(3)
- – 6.1(1), Note 1
- ● 6.1(1), Note 2B
- – 6.3.2.2(2)
- ● 6.3.2.3(1)
- ● 6.3.2.3(2)
- – 6.3.2.4(1)B, Note 2B
- – 6.3.2.4(2)B
- ● 6.3.3(5), Note 2
- ● 6.3.4(1)
- ● 7.2.1(1)B
- ● 7.2.2(1)B
- ● 7.2.3(1)B
- ● C.2.2(3), Note 1
- ● C.2.2(4)
- – BB.1.3(3).

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

Actions and environmental influences

2.3.1(1), Note 1
The characteristic values of ice loads are determined according to standard ISO 12494.

General

3.1(2)
In addition to the materials given in Table 3.1, the following steel grades may also be used on the conditions presented below:

a) Steel grades S315MC, S355MC, S420MC and S460MC according to standard SFS-EN 10149-2.

b) Steel grades S260NC, S315NC, S355NC and S420NC according to standard SFS-EN 10149-3.

In the cases a) and b) the requirement for the fracture toughness should be determined according to option 5 in section 11 of standard SFS-EN 10149-1.

The value of β_w for steel grades pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3 is given in the National Annex to standard SFS-EN 1993-1-8.

For steel grades pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3, mechanical properties at elevated temperatures may be determined according to the National Annex to standard SFS-EN 1993-1-2.

For steel grades pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3, for the avoidance of brittle failure, the maximum permissible values of element thickness may be determined according to the National Annex to standard SFS-EN 1993-1-10.

Other steel grades may be used if the properties of the steel grade and the compatibility of the properties with the design conditions in the Eurocode SFS-EN 1993 and its National Annexes have been reliably demonstrated.

Material properties

3.2.1(1)
Both alternatives may be used.

Ductility requirements

3.2.2(1)
Steels used should fulfil the requirements given in the Note, if not otherwise mentioned in some part of standard SFS-EN 1993 or in the other National Annexes to standard SFS-EN 1993.

Fracture toughness

3.2.3(1)
The lowest service temperature is determined according to the Ministry of Environment Decree (8/16) on the application of standard SFS-EN 1991-1-5. The fracture toughness should be checked at all operating temperatures with a relevant load case corresponding to that temperature. The situation during erection stage should also be taken into account by using appropriate load combinations and temperatures during erection.

3.2.3(3)B
Clause 2.1(2) of standard SFS-EN 1993-1-10 states that fracture toughness need not be specified for elements only in compression. However, the value $\sigma_{Ed} = 0.25\, f_y(t)$ is applied when determining the maximum allowed thickness referred to in Table 2.1 of standard SFS-EN 1993-1-10.

Structural stability of frames

5.2.2(8)
Buckling lengths should be determined according to the rules of structural mechanics.

When this method is used, the second order effects should be taken into account in the design of cross-section resistance of members and in the design of joints, connections and splices.

Imperfections for global analysis of frames

5.3.2(11), Note 2
The presented method is not used.

General

6.1(1), Note 2B
The prerequisite for the use of partial factor $\gamma_{M1} = 1.0$ for the resistance of the member is, that the buckling curve used with methods presented in clause 6.3.1 and

6.3.4 in standard SFS-EN 1993-1-1 is selected according to clause 6.3.4(1) of this instruction.

If a deviation is made from the tolerances underlying the design rules into a direction that is unfavourable in terms of structural durability, calculations are used to indicate that the confidence interval required by standard SFS-EN 1993 and its National Annexes is achieved.
When the initial curvature assumed in the design is higher than L/1000, the member is treated as a compressed and bent, in which case the increase in bending moment ΔM_{Ed} *caused by exceeding the initial curvature of L/1000 at the point of highest bending* v_{design} *is calculated from the following expression:*

$$\Delta M_{Ed} = N_{Ed}(v_{\mathrm{desing}} - L/1000) \qquad (1.1)$$

where: v_{design} *corresponds to the initial curvature assumed in the design.*

The integral tolerance concerning the initial curvature of a compressed rod pursuant to standard SFS-EN 1090-2 is normally L/1000 and occasionally L/750.

In accidental limit states, with the exception of fire, the same design expressions and conditions and partial material factors are used as in the normal temperature design, except for γ_{M2} = 1.1.

Tension

6.2.3
The design value of the tension resistance of a blind tensioned cross-section is calculated according to expression (6.6) of standard SFS-EN 1993-1-1.

Lateral torsional buckling curves for rolled sections or equivalent welded sections

6.3.2.3(1)
When determining the lateral torsional buckling curves, the following values are used for non-dimensional slenderness during buckling, $\lambda_{\mathrm{LT,0}}$ and the parameter β:

a) For rolled double symmetrical I-sections and H-sections and hot-finished and cold-formed hollow sections with a constant cross-section:

   $\bar{\lambda}_{\mathrm{LT,0}}$ = 0.4

   β = 0.75

b) For welded double symmetrical I-sections with a constant cross-section:

$\bar{\lambda}_{LT,0} = 0.2$

$\beta = 1.0.$

In both cases, the lateral torsional buckling curve is selected from Table 1.

Table 1. Selection of lateral torsional buckling curve for cross-sections using equation (6.57) from the standard. In the table, *h* is the height of the cross-section and *b* is the width.

Cross-section (constant cross-section)	**Limits**	**Buckling curve**
Rolled double symmetrical I- and H-sections and hot finished hollow sections	$h/b \leq 2$ $2 < h/b < 3.1$	b c
Welded double symmetrical I-sections and cold-formed hollow sections	$h/b \leq 2$ $2 < h/b < 3.1$	c d

In all other cases, the rules given in clause 6.3.2.2 of standard SFS-EN 1993-1-1 should be applied.

6.3.2.3(2)
The value $f = 1.0$ is adopted for the conversion factor of the reduction factor for lateral torsional buckling resistance.

Uniform members in bending and axial compression

6.3.3
Method 2 pursuant to Annex B of standard SFS-EN 1993-1-1 is also applied to circular structural hollow sections.

6.3.3(5), Note 2
The alternative method 2 should be used, if applicable. The alternative method 1 may be used.

General method for lateral and lateral torsional buckling of structural components

6.3.4(1)
This method may be used when other methods given in standard SFS-EN 1993-1-1 are not applicable. In these cases the applicability of the general method should be verified case by case.

The prerequisite for using the partial material factors pursuant to this National Annex is that the lateral buckling curves are selected as follows according to Table 2:

Table 2.. Selection of lateral buckling curve used In context of methods presented in clause 6.3.1 and in this clause 6.3.4 of standard SFS-EN 1993-1-1 relating to z-z axis for the steel grade S460 rolled I-profiles on the basis of height /width ratio of the member and the thickness of the flange, change to the table 6.2 of standard SFS-EN 1993-1-1.

Limits	Flange thickness	Lateral buckling about this particular axis	Lateral buckling curve
h/b>1.2	$t_f \leq 40$ mm	z-z	curve “a” instead of “a_0”
h/b>1.2	40 mm < $t_f \leq 100$ mm	z-z	curve “b” instead of “a”
h/b≤1.2	$t_f \leq 100$ mm	z-z	curve “b” instead of “a”

Vertical deflections

7.2.1(1)B
The final vertical (w_{max}, see standard SFS-EN 1990) and horizontal deflections due to characteristic load combinations calculated with a static load should not exceed the values in Table 3, if the deflections are harmful, unless other values are determined to be more suitable due to type of structure, use or the nature of the activity. Precamber (w_c, see standard SFS-EN 1990) may be used for compensation of the deflection of the permanent load unless harm is caused by it.

Table 3. Serviceability limit states for deflections

Structure	Serviceability limit state for deflection limits
Main girders: - roofs - floors Cantilevers	 *L*/300 *L*/400 *L*/150
Roof purlins	*L*/200
Wall purlins	*L*/150
Sheetings: - in roofs with no risk for accumulation of water or other risk for failure of the roof - in roofs with risk for accumulation of water or other risk for failure of the roof - when $L \leq$ 4.5 m - when 4.5 m < $L \leq$ 6,0 m - when L > 6.0 m - in floors - in walls - cantilevers	 *L*/100 *L*/150 30 mm *L*/200 *L*/300 *L*/100 *L*/100
Horizontal deflection of the structure - 1- and 2-storey high buildings - other buildings	 *H*/150 *H*/400
L is the span *H* is the height of the building at the point of local application	
For buildings supporting crane gantry girders, see standard SFS-EN 1993-6 and its National Annex.	

Horizontal deflections

7.2.2(1)B
Horizontal deflection limits are presented in Table 3 of clause 7.2.1(1)B of this National Annex.

Dynamic effects

7.2.3(1)B
In order to account for dynamic effects, the NCCI 1 document that is appended to the instructions concerning national choices for standard SFS-EN 1993-1-1 and contains non-contradictory information is applied.

Annex C
Selection of execution class

This Annex replaces Annex B to standard SFS-EN 1090-2.

C.2.2(3), Note 1

The execution class is selected based on Table C.1 of standard SFS-EN 1993-1-1. The consequences classes pursuant to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are used in the selection of the execution class.

Normally, the execution class selected for an individual structural component (construction element component) is the same as the execution class for the entire structure. However, if the consequences class or load type of a specific structural component (construction component) or detail differs from the consequences class or load type of the entire structure, the execution class is selected on the basis of the consequences class for the fabricated product or detail of the component or the load type it is subjected to.

C.2.2(4)

In the following cases, the requirements set for execution class 2 (EXC2) in standard SFS-EN 1090-2 are adopted, at a minimum, even if the structure is in execution class 1 (EXC1):

a) welded structural components manufactured from steel products with a grade higher than S355
b) welded fabricated products that are important for structural functionality and installed at the site by means of welding
c) structural component made of circular structural hollow sections by welding and where pipe ends need to be cut to a specific shape if the cutting is done manually.
d) structural component that are manufactured by means of hot forming or that are heat treated during manufacture. Work instructions pursuant to standard SFS-EN 1090-2 or SFS-EN 1090-4 are prepared for structural components that are manufactured by means of hot forming or that are heat treated during manufacture or hot straightened.

NCCI 1 for standard SFS-EN 1993-1-1: Floor vibration design

Scope and symbols

This instruction presents the calculation method for determining the acceptability of vibration caused by walking on light and heavy floors.

The following symbols are used:

a *is the calculated acceleration resulting from humans walking [m/s^2];*
x *is the maximum width or length of the room [m];*
b *is the width of the floor [m];*
b_{eff} *is the effective width of the vibrating floor part [m];*
e *is the Neper number (= 2.718);*
s *is the distance between floor beams [m];*
f_0 *is the floor's fundamental frequency [Hz];*
l *is the length of the floor beams [m];*
m *is the mass of the entire floor per unit of surface area + imposed load share of 30 [kg/m^2];*
L *is the span of the main support [m];*
El *is the reduced modulus of elasticity corresponding to the floor's longitudinal direction l [N/m^2];*
Il *is the bending stiffness corresponding to the floor's longitudinal direction l, calculated by unit of width [m^4/m];*
(EI)b *is the floor's lower stiffness corresponding to the lateral direction b, Eb .Ib [Nm^2/m];*
(EI)l *is the floor's higher stiffness corresponding to the longitudinal direction l, El .Il [Nm^2/m];*
(EI)L *is the stiffness of the floor's main supports EL .IL [Nm^2/m];*
W *is the effective mass of the floor part involved in the vibration [kg];*
P *is the weight of the person causing the vibration [N];*
R *is the acceleration reduction factor (= 0.7) [-];*
δ_0 *is the maximum overall deflection caused by a point force of 1 kN [m];*
δ_1 *is the maximum local deflection caused by a point force of 1 kN [m];*
ζ *is the damping ratio [-].*

Limitations of the method

These instructions are subject to the following prerequisites:

- *the floor is a part of residential or office facilities*
- *the fundamental frequency of the floor is above 3 Hz*
- *the vibration is caused by persons walking*
- *there are no special requirements for the amplitude of the vibration.*

The method shall not be used for exercise and sports facilities where the load and requirement levels differ from the above, or in facilities where vibration is caused by machinery.

General

Vibration caused by walking may become harmful if the periodic load components of walking are excessively amplified due to resonance, if the impact of the heel on the floor causes excessively large vibration or if the floor bends excessively under the walker.

Resonance is considered a determining factor in the design if the fundamental frequency of the floor vibration is lower than 10 Hz. If the frequency is higher than this, the bending or vibration of the floor will be the determining factor in the dimensioning. Due to the change in the determining factor, there is a discontinuity in the design at the 10 Hz point. Low natural frequencies are typical of heavy floors, while light floors have high natural frequencies.

This presents the instructions for the vibration classification of floors and describes the vibration analysis for a rectangular floor. The studied floor may also be a part of a larger floor (Figure 1).

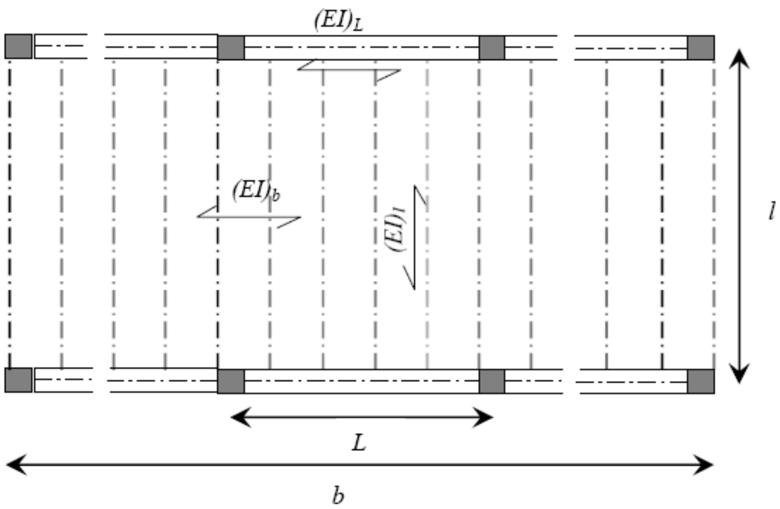


Figure 1. *A typical floor section that comprises the surface slab, floor beams and main supports.*

Vibration criteria

The following are taken into account in the floor analysis:

- *Total floor frame deflection $_0$ caused by a local load of 1 kN when the natural frequency of the floor is above 10 Hz. Such floors are referred to as high-frequency floors.*

- *The acceleration a in the floor frame structure caused by a single person walking when the natural frequency of the floor is below 10 Hz. Such floors are referred to as low-frequency floors.*
- *Local floor surface deflection $_1$ caused by a local load of 1 kN. Local deflection concerns floor surface deflection between the floor beams, floating floors and raised floors.*

Floors are divided into vibration classes pursuant to Table 1. The limits set for the floor frame structure given in Table 1 may be increased by the following factor:

$$k = \frac{1}{0{,}318 + 0{,}114 \cdot x} \quad (1.1)$$

when the maximum length or width of the floor x is below 6 m. When x $\geq$ 6 m, the value to be used is k = 1.0. A floor belonging to a specific class must meet both the criterion concerning the floor frame and the criterion concerning local deflection.

Table 1 presents the vibration classification for the floors and table 2 presents the vibration class's area of application for residential and office buildings.

Table 1. *Floor vibration classification*

	Criterion for floor frame		Criterion for local deflection[1)]
Vibration class	High-frequency floors	Low-frequency floors	Both high- and low-frequency floors
A	$\delta_0 < 0.12$ mm	$a < 0.03$ m/s^2	$\delta_1 < 0.12$ mm
B	$\delta_0 < 0.25$ mm	$a < 0.05$ m/s^2	$\delta_1 < 0.25$ mm
C	$\delta_0 < 0.50$ mm	$a < 0.075$ m/s^2	$\delta_1 < 0.50$ mm
D	$\delta_0 < 1.0$ mm	$a < 0.12$ m/s^2	$\delta_1 < 1.0$ mm
E	$\delta_0 < 1.0$ mm	$a > 0.12$ m/s^2	$\delta_1 > 1.0$ mm

1) *The local surface slab deflection is checked when the surface slab has a span of over 600 mm.*

Table 2. Vibration class's area of application for residential and office buildings

Vibration class	Vibration class's area of application
A	Normal class for vibration that transfers from one tenement to another. A special class when the cause of the vibration is inside the same tenement.
B	Lower class for vibration that transfers from one tenement to another. Higher class for residential and office buildings when the cause of the vibration is inside the same tenement.
C	Normal class for residential and office buildings when the cause of the vibration is inside the same tenement.
D	Lower class for residential buildings when the cause of the vibration is inside the same tenement. For example, attics in detached houses or holiday houses.
E	A class where no limitations are set.

Natural frequency of the floor

The fundamental frequency of a simple rectangular floor supported on four sides is calculated with the expression:

$$f_0 = \frac{\pi}{2l^2}\sqrt{\frac{(EI)_l}{m}} \cdot \sqrt{1 + \left[2\left(\frac{l}{b}\right)^2 + \left(\frac{l}{b}\right)^4\right]\frac{(EI)_b}{(EI)_l}} \tag{1.2}$$

where l is the length of the floor, $(EI)_l$ is the floor's higher stiffness corresponding to the longitudinal direction l, $(EI)_b$ is the floor's lower stiffness corresponding to the lateral direction b and m is the mass of the entire floor per unit of surface area. Thirty, 30, kg/m^2 of the imposed load is included in the mass of the floor.

Commonly, the supports of the floor edges in the direction of the floor beams do not affect the natural frequency. In this case, natural frequency may be calculated from the following expression:

$$f_0 = \frac{\pi}{2l^2}\sqrt{\frac{(EI)_l}{m}}, \tag{1.3}$$

Expression (1.3) underestimates the natural frequency by a maximum of 5%, when b/l >1.0 and $(EI)_l/(EI)_b$ >30, but if b/l = 0.5, the same precision can only be reached once $(EI)_l/(EI)_b$ >200.

If the floor beams (length l) are supported by the main supports (length L=b), the system's fundamental frequency is calculated by using the natural frequencies of the floor beam and main support in the following expression:

$$f_0 = \frac{1}{\sqrt{\frac{1}{f_{0,l}^2} + \frac{1}{f_{0,L}^2}}} \tag{1.4}$$

where $f_{0,l}$ is calculated from the expression (1.5) and the natural frequency of the main support is calculated from the following expression:

$$f_{0,L} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{m}}\ . \tag{1.5}$$

The factor $(EI)_L$ is the common bending stiffness of the main supports and surface slab per unit of length.

Calculating total deflection

The total floor deflection $_0$ caused by a local load of 1 kN is checked when the natural frequency of the floor is above 10 Hz.

Deflection is calculated as the deflection of a rectangular orthotropic slab that is supported on four sides. The deflection at the centre point of the slab caused by point load F = 1 kN is calculated from the following expression:

$$\delta_0 = \gamma \cdot \frac{Fl^2}{(EI)_l} \quad \text{, where} \tag{1.6}$$

$$\gamma = \frac{4}{\alpha\pi^4}\sum_i\sum_j \frac{1}{(2i-1)^4 + \beta\left(\frac{2j-1}{\alpha}\right)^4}\ ; \quad \alpha = \frac{b}{l} \ \text{ ja } \ \beta = \frac{(EI)_b}{(EI)_l} \tag{1.7}$$

In many cases, the supports of the floor edges in the direction of the floor beams do not affect the deflection. In this case, expression (1.7) may be replaced by the following expression:

$$\gamma = \frac{1}{42 \cdot \left[\frac{(EI)_b}{(EI)_l}\right]^{1/4}} \tag{1.8}$$

The difference between the results of expressions (1.7) and (1.8) is at most 2.5%, when b/l >1.0 and $(EI)_l/(EI)_b$ >20, but if b/l = 0.5, the same precision can only be achieved once $(EI)_l/(EI)_b$ >300.

If the deflection calculated with expression (1.6) is higher than the deflection of a replacement beam separated from the floor at point load F = 1 kN, the reference deflection to be used is the maximum deflection calculated with a compensation beam:

$$\delta_{\max} = \frac{Fl^3}{48 \cdot s \cdot (EI)_l}$$ *, where* (1.9)

s is the distance between floor beams.

If the floor beams are supported by the main supports, the deflection of the main supports is added to the deflection.

Calculating acceleration

The floor acceleration caused by a single person walking is checked when the natural frequency of the floor is below 10 Hz. Acceleration is calculated from the following expression:

$$a = \frac{R \cdot P}{W \cdot \varsigma} \cdot 0{,}83 \cdot e^{-0{,}35 f_0}$$ *, where* (1.10)

P=800 N (weight of the walker), R = 0.7 and e = 2.718. The Normally value ζ = 0,03 is used for the damping ratio. However, if floor construction contain little non load bearing structures (internal walls, ceilings, ducts, furnitures and so on), value ζ = 0,02 is used for the damping ratio.

The effective mass involved in the vibration of a rectangular floor supported on four sides is calculated with the following expression:

$W = m \cdot b_{eff} \cdot l$ *, where:* (1.11)

$$b_{eff} = 2{,}0 \cdot \left[\frac{(EI)_b}{(EI)_l} \right]^{1/4} \cdot l$$ (1.12)

however, b_{eff} may be at most 2/3 of the total floor width transverse to the floor beams.

If the rectangular floor is unsupported along its other edge in the direction of the floor beam, a factor of 1.0 will be used instead of 2.0 in expression (1.12).

If the floor beams (length l) are supported by the main supports (length L), the effective mass involved in the vibration is calculated with the following expression:

$$W = \frac{W_l}{1 + f_{0,l}^2 / f_{0,L}^2} + \frac{W_L}{1 + f_{0,L}^2 / f_{0,l}^2} \ ,$$ (1.13)

where W_l is received from expressions (1.11) and (1.12).

Factor

$$W_L = m \cdot l_{eff}\ L \qquad \text{, where} \qquad (1.14)$$

$$l_{eff} = 1,6 \cdot \left[\frac{(EI)_l}{(EI)_L}\right]^{1/4} \cdot L \qquad (1.15)$$

however, l_{eff} may be at most 2/3 of the total floor width transverse to the main supports. If the main support is located at the free edge of the floor, floor stiffness $(EI)_L$ is reduced by 50 per cent.

Calculating local deflection

Local deflection, $_{1,}$ concerns floor surface deflection between the floor beams, floating floors and raised floors. Local deflection is the difference between the deflection at the 1 kN point load and the deflection at a distance of 600 mm (image 2). The deflection of the floor beams does not need to be taken into account.

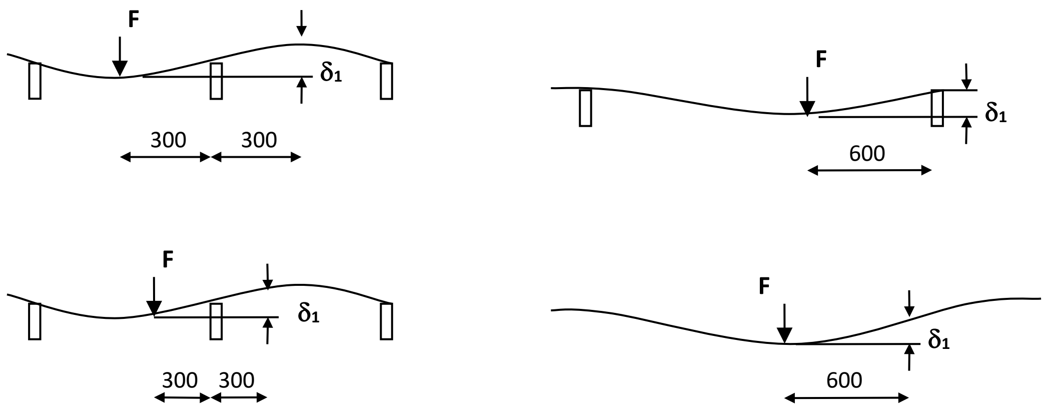


Figure 2. *Examples of deflection in the floor surface structure*

National Annex to standard SFS-EN 1993-1-2 Part 1-2: General rules. Structural fire design

As regards standard SFS-EN 1993-1-2, the recommended values set forth in standard SFS-EN 1993-1-2 and all the annexes to standard SFS-EN 1993-1-2 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 1993-1-2:
- 2.3(1)
- 2.3(2)
- • 4.1(2)
- • 4.2.3.6(1), Note 2
- 4.2.4(2).

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

Scope of Standard EN 1993-1-2

1.1.2(6)
The instructions in standard SFS-EN 1993-1-2 and its National Annex may be used also for steels given in clause 3.1(2) of National Annex to standard SFS-EN 1993-1-1.

Parametric fire exposure

2.1.3
As regards the separating function, standard SFS-EN 1994-1-2 and its National Annex may be used.

Member analysis

2.4.2(3)
When using the partial factors from standard SFS-EN 1990 and the Ministry of Environment Decree 3/16 concerning its application, Figure 2.1 in standard SFS-EN 1993-1-2 will change as presented in Figure 1.

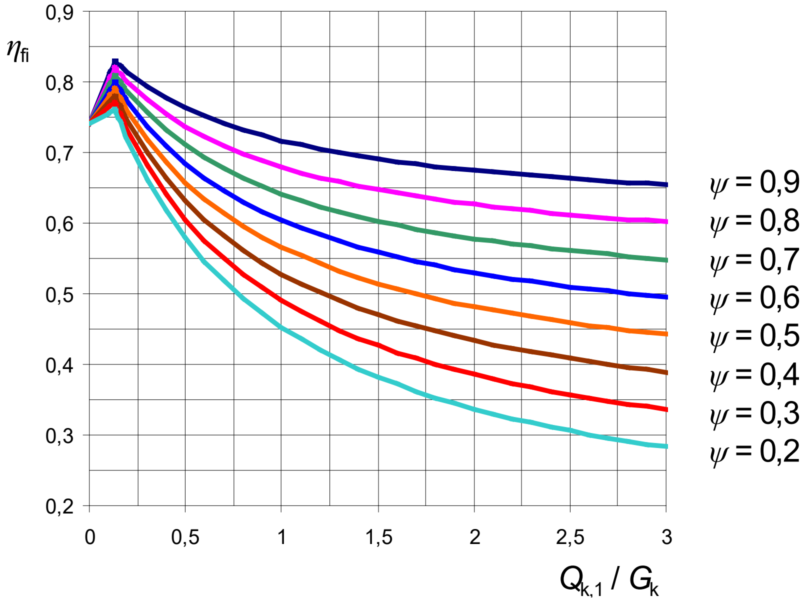


Figure 1. *The variation of the reduction factor η_{fi} as a function of the load ratio of the nominal values of dominant variable action and permanent action $Q_{k,1}$ / Gk according to the load combination rules presented in the Ministry of Environment Decree 3/16 concerning the application of standard SFS-EN 1990.*

2.4.2(3), Note 2

Approximate values are not used.

General

4.1(2)

Advanced calculation methods may be used.

Members with Class 4 cross-sections

4.2.3.6(1), Note 2

For the critical temperature of steel, a value of θ_{crit} = 450°C is adopted together with its corresponding value $k_{p0,2,\theta} = 0{,}59$. $k_{p0,2,\theta}$, see Annex E.

Annex C

Stainless steel

Mechanical properties of steel grades 1.4318, 1.4318 C850 and 1.4571 C850 at elevated temperatures

Table 1 presents the reduction factors for strength and modulus of elasticity in proportion to a value corresponding to a temperature of 20 ℃ for the determination of *the stress/strain relationships of stainless steel grades 1.4318, 1.4318 C850 and 1.4571 C850 at elevated temperatures, and parameter $g_{2,\theta}$. These reduction factors are:*

$k_{0.2\mathrm{proof},\theta}$ — *the 0.2 limit at a temperature of θ in relation to a design strength corresponding to a temperature of 20℃, in other words* $f_{0,2\mathrm{proof},\theta}/f_{\mathrm{y}}$

$g_{2,\theta}$ — *parameter at a temperature of θ for calculating strength $f_{2,\theta}$ corresponding to a total elongation of 2% according to expression (1.1):*

$$f_{2,\theta} = f_{0,2\mathrm{proof},\theta} + g_{2,\theta}(f_{\mathrm{u},\theta} - f_{0,2\mathrm{proof},\theta}) \qquad (1.1)$$

$k_{\mathrm{u},\theta}$ — *ultimate tensile strength at a temperature of θ in relation to a value corresponding to a temperature of 20℃, in other words* $f_{\mathrm{u},\theta}/f_{\mathrm{u}}$

$k_{\mathrm{E},\theta}$ — *linear initial modulus of elasticity at a temperature of θ in relation to a value corresponding to a temperature of 20℃, in other words E_θ/E*

where:

E — *is the modulus of elasticity at a temperature of 20°C (= 200,000 N/mm^2)*

f_{y} — *is the nominal value for yield strength at a temperature of 20°C according to clause 3.2.4 of standard SFS-EN 1993-1-4*

f_{u} — *is the nominal value for tensile strength at a temperature of 20°C according to clause 3.2.4 of standard SFS-EN 1993-1-4*

Table 1. Reduction factors and parameter $g_{2,\theta}$ for strength and modulus of elasticity at elevated temperatures

Temperature θ(°C)	*Reduction factor $k_{0.2proof,2}$*	*Parameter $g_{2,\theta}$*	*Reduction factor $k_{u,\theta}$*	*Reduction factor $k_{E,\theta}$*
Steel grade 1.4318				
20	*1.00*	*0.25*	*1.00*	*1.00*
100	*0.78*	*0.25*	*0.74*	*0.96*
200	*0.65*	*0.25*	*0.73*	*0.92*
300	*0.57*	*0.25*	*0.64*	*0.88*
400	*0.51*	*0.25*	*0.60*	*0.84*
500	*0.48*	*0.25*	*0.55*	*0.80*
600	*0.46*	*0.27*	*0.52*	*0.76*
700	*0.40*	*0.27*	*0.40*	*0.71*
800	*0.27*	*0.26*	*0.26*	*0.63*
Steel grade 1.4318 C850				
20	*1.00*	*0.21*	*1.00*	*1.00*
100	*0.86*	*0.24*	*0.71*	*0.91*
200	*0.77*	*0.25*	*0.61*	*0.88*
300	*0.69*	*0.24*	*0.60*	*0.84*
400	*0.68*	*0.24*	*0.57*	*0.80*
500	*0.65*	*0.25*	*0.53*	*0.76*
600	*0.54*	*0.25*	*0.45*	*0.72*
700	*0.40*	*0.26*	*0.34*	*0.67*
800	*0.23*	*0.25*	*0.24*	*0.52*
900	*0.11*	*0.25*	*0.10*	*0.35*
Steel grade 1.4571 C850				
20	*1.00*	*0.36*	*1.00*	*1.00*
100	*0.96*	*0.36*	*0.94*	*0.96*
200	*0.95*	*0.36*	*0.88*	*0.92*
300	*0.92*	*0.36*	*0.84*	*0.88*
400	*0.89*	*0.36*	*0.82*	*0.84*
500	*0.83*	*0.36*	*0.79*	*0.80*
600	*0.81*	*0.36*	*0.72*	*0.76*
700	*0.60*	*0.37*	*0.53*	*0.71*
800	*0.35*	*0.39*	*0.38*	*0.63*
900	*0.10*	*0.40*	*0.20*	*0.45*

National Annex to standard SFS-EN 1993-1-3 Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting

As regards standard SFS-EN 1993-1-3, the recommended values set forth in standard SFS-EN 1993-1-3 and all the annexes to standard SFS-EN 1993-1-3 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 1993-1-3:

– 2(3)P
– 2(5)
• 3.1(3), Note 1
• 3.1(3), Note 2
– 3.2.4(1)
– 5.3(4)
– 8.3(5)
• 8.3(13), Table 8.1
• 8.3(13), Table 8.2
• 8.3(13), Table 8.3
• 8.3(13), Table 8.4
– 8.4(5)
– 8.5.1(4)
– 9(2), Note 1
– 10.1.1(1)
– 10.1.4.2(1)
– A.1(1), Note 2
– A.1(1), Note 3
• A.6.4(4)
– E.(1).

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

Basis of Design

2(6)
Sheeting structures in structural classes III and II are in consequences class CC1.

Sheeting structures in structural class I are in consequences class CC1 when they are subjected to actions perpendicular to the surface that cause bending. This does not apply to loads which are induced when sheeting is used to transfer shear forces parallel to the surface of the sheeting or normal forces.

General

3.1(3), Note 1
The recommended value should be used if it can not be shown that the values given in Table 3.1a of standard SFS-EN 1993-1-3 are reached in the rolling direction and perpendicular to the rolling direction.

3.1(3), Note 2
Steels pursuant to Table 3.1b of standard SFS-EN 1993-1-3 may be used. If higher values than given in Table 3.1b of standard SFS-EN 1993-1-3 are used for steels pursuant to standard SFS-EN 10346, fulfilment of the strength values should be verified by means of material certificates that are valid for the material used.

The rules according to standard SFS-EN 1993-1-3 may be applied for steels pursuant to standard SFS-EN 10025-5. The rules according to standard SFS-EN 1993-1-3 may be applied to steels pursuant to standard SFS- EN 10025-6, if the limitations given in standards SFS-EN 1993-1-3 and SFS-EN 1993-1-12 are taken into account.

Durability

4
The expected service life of a metal sheet is affected by the environmental corrosion exposure, the sheet's corrosion protection and the care, service and maintenance activities carried out during use. The first inspection of the protection will be made at the latest after 10 years and at least every 5 years thereafter. In climate exposure classes C4-C5, the protection is inspected every 2–3 years.

The instructions for selecting corrosion protection for a continuously hot-dip-galvanised sheet that is exposed to climate are presented in Table 1.

Table 1. Selection of corrosion protection for the steel sheet**.**

<table>
<tr><td>Climate exposure class,
SFS-EN ISO 12944-2</td><td>Corrosion protection</td></tr>
<tr><td>C2</td><td>Z350[1] or coil-coated[2] Z275[1]</td></tr>
<tr><td>C3</td><td>Z350[1] + paint[3]or coil-coated[2]Z275[1]</td></tr>
<tr><td>C4</td><td>Z350[1] + paint[3]or coil-coated[4]Z275[1]</td></tr>
<tr><td>C5</td><td>Selected for each case</td></tr>
<tr><td colspan="2">1) Sheet that has been continuously hot-dip-galvanised on the line (EN 10346), nominal thicknesses: Z275 = 20 µm/side and Z350 = 25 µm/side or a metal coating that has similar corrosion resistance properties to the above, such as ZA or AZ coating if its thickness corresponds to the Z coating (Z275=20 µm, ZA255=20 µm, AZ150= 20 µm).

2) An organic coil-coating that contains a corrosion prevention primer that is coated continuously on the line (EN 10169): polyester, polyurethane or PVDF coating with a thickness not less than 25 µm or a coil-coating whose corrosion resistance corresponds to the above.

3) Painting at the worksite requires a chemical pre-treatment of the metal coating. A mechanical cleaning process that damages the metal coating must not be used.
- Painting a continuously hot-dip-galvanised (Z350) sheet is not necessary in climate corrosivity category C2. If the sheet is painted, the expected service interval is 5–10 years when the total film thickness of the coating is generally not less than 40 µm and 10–20 years when the film thickness of the painting is generally not less than 80 µm.
- In climate corrosivity category C3, respectively, the total film thickness for the painting is generally not less than 80/120 µm.
- In climate corrosivity category C4, respectively, the total film thickness for the painting is generally not less than 120/160 µm.

The requirements set for corrosion prevention painting are given in the execution specification. The instructions concerning the paint combinations and the general quality requirements for painting are followed in the painting work.

4) An organic coil-coating that contains a corrosion prevention primer that is coated continuously on the line (EN 10169): polyester, polyurethane or PVDF coating with a thickness not less than 35 µm or a coil-coating whose corrosion resistance corresponds to the above.</td></tr>
</table>

Note 1.
When other coating types and/or thicknesses are used, their corrosion resistance is determined on a per-case basis.

Note 2.
Under conditions involving chemical exposure (particularly to corrosive chemicals), the selection and adequacy of corrosion protection are determined on a per-case basis.

General

7.1(1)
Guidance given in the National Annex of standard SFS-EN 1993-1-1 should be applied. For crane supporting structures, see National Annex of standard SFS-EN 1993-6.

Connections made with mechanical fasteners

8.3(13), Table 8.1

If the properties of the blind rivet have been reliably determined, values pursuant to Table 2 may be adopted. In this case, the design tensile resistance for the rivet should be calculated as follows:

$F_{t,Rd} = F_{v,Rd} = F_{v,Rk} / \gamma_{M2}$ (1.1)

Table 2. Characteristic shear strengths for blind rivet $F_{v,Rk}$ (N/rivet)

Diameter of the shank (mm)	Material of the rivet [1)]			
	Steel	Stainless steel	Monel-metal [2)]	Aluminium
4.0	1600	2800	2400	800
4.8	2400	4200	3500	1100
5.0	2600	4600	-	-
6.4	4400	-	6200	2000

1) According to the applied standard or a reliable analysis.

2) Nickel-copper alloy containing two parts nickel and one part copper.

If the properties of the blind rivet have been reliably determined, values higher than those given in Table 2 may also be used if they are based on tests whose results have been analysed pursuant to the Annex D of standard SFS-EN 1990.

In addition, the rules given in Annex A of SFS-EN 1993-1-3 should be taken into account, if appropriate.

8.3(13), Table 8.2

If the properties of the self-tapping and self-drilling screws have been reliably determined, values pursuant to Table 3 may be adopted. In this case, the design tensile resistance for the screw should be calculated as follows:

$F_{t,Rd} = 1.2 \cdot F_{v,Rd} = 1.2 \cdot F_{v,Rk} / \gamma_{M2}$ (1.2)

Table 3. Characteristic shear strengths for shear forming self-tapping screws and for self-drilling self-tapping screws $F_{v,Rk}$ (N/screw)

The outer diameter of the thread (mm)	Material of the screw [1]	
	Hardened steel	Stainless steel
4.8	5200	4600
5.5	7200	6500
6.3	9800	8500
8.0	16300	14300
[1] According to the applied standard or a reliable analysis.		

If the properties of the self-tapping and self-drilling screws have been reliably determined, values higher than those given in Table 3 may also be used if they are based on tests whose results have been analysed pursuant to the Annex D of standard SFS-EN 1990.

In addition, the rules given in Annex A of SFS-EN 1993-1-3 should be taken into account, if appropriate.

8.3(5), Table 8.3
Values that have been reliably determined may be used for the shear resistance, put-out resistance and tension resistance of the cartridge fired pins.

8.3(5), Table 8.4
Values that have been reliably determined may be used for the put-out resistance of the screws.

Design values

A.6.4(4)
The partial factor should be determined based on testing according to Annex D of SFS-EN 1990. In addition, the rules given in Annex A of SFS-EN 1993-1-3 should be used, if applicable. If only the characteristic value without a design formula is determined based on testing, then the recommended values for γ_M should be used.

Annex E
Simplified design for purlins

Annex E is not used.

National Annex to standard SFS-EN 1993-1-4 Part 1-4: General rules. Supplementary rules for stainless steel

As regards standard SFS-EN 1993-1-4, the recommended values set forth in standard SFS-EN 1993-1-4 and all the annexes to standard SFS-EN 1993-1-4 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 1993-1-4:
- 2.1.4(2), Note 2
- 2.1.5(1)
- 5.1(2)
- 5.5(1), Note 1
- 5.5(1), Note 2
- • 5.6(2)
- 6.1(2), Note 2
- 6.2(3).
- 7(1)
- A.2(8)
- A.3, Table A.4

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

Scope

When applying the non-contradictory complementary information to standard SFS-EN 1993-1-8 to overlap type joints in structural hollow sections made of stainless steel, ultimate strength f_u is replaced by the yield strength f_y.in given equations.

Shear resistance

5.6(2)
When determining shear resistance, the value η = 1.20 should be used when the 0.2% proof strength of steel is not higher than 460 MPa and when the temperature of steel is not more than 400°C. When the temperature of steel is greater than 400 °C, the value η = 1.00 should be used.

Annex C
Modelling of material behaviour

Mechanical properties at elevated temperatures for steel grades 1.4318, 1.4318 C850 and 1.4571 C850, see the National Annex to standard SFS-EN 1993-1-2.

National Annex to standard SFS-EN 1993-1-5 Part 1-5: Plated structural elements

As regards standard SFS-EN 1993-1-5, the recommended values set forth in standard SFS-EN 1993-1-5 and all the annexes to standard SFS-EN 1993-1-5 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 1993-1-5:
– 2.2(5), Note 1
• 3.3(1), Note 1
– 4.3(6)
• 5.1(2), Note 2
– 6.4(2)
– 8(2)
– 9.1(1)
– 9.2.1(9)
• 10(1), Note 2
– 10(5) Note
• C.2(1)
– C.5(2), Note 1
– C.8(1), Note 1
– C.9(3)
– D.2.2(2).
A national choice has been made in the clauses marked •.

Shear lag phenomenon at ultimate limit state

3.3(1), Note 1
The method given in the Note 3 should be used if not otherwise specified in the standards SFS-EN 1993- 2...SFS-EN 1993-6 and in their National Annexes.

Grounds

5.1(2), Note 2
The recommended values are followed when the temperature of the steel is at most 400°C. When the steel temperature is higher than 400°C, the value used is $\eta = 1.00$.

Interaction

Section 7

Standard SFS-EN 1993-1-5 does not provide instructions for the interaction of shear force and point load at ultimate limit state. The following instructions are followed for the interaction of shear force and point load.

The interaction of shear force and point load at ultimate limit state is checked with the condition:

$$\left(\frac{V_{Ed} - 0,5F_{Ed}}{V_{b,Rd}}\right)^a + \left(\frac{F_{Ed}}{F_{Rd}}\right)^b \leq 1,0 \qquad (1.1)$$

where:

V_{Ed}	*is the design value for shear force pursuant to standard SFS-EN 1993-1-5*
F_{Ed}	*is the design value for transverse point load pursuant to standard SFS-EN 1993-1-5*
$V_{b,Rd}$	*is the design value for web shear resistance that is determined according to standard SFS-EN 1993-1-5, taking into account the National Annex to standard SFS-EN 1993-1-5*
F_{Rd}	*is the design value for web shear resistance for transverse loads in relation to buckling that is determined according to standard SFS-EN 1993-1-5, taking into account the National Annex to standard SFS-EN 1993-1-5*
a	*1.6*
b	*1.0.*

The method is suitable for double symmetrical I-profiles with an unbraced web or a longitudinally braced web and the steel grade of the entire profile is at most S355. The value η = 1 is adopted for the variable η in standard SFS-EN 1993-1-5. The method is suitable for all cross-section classes. The method is not suitable for hybrid structures.

Reduced tension method

10(1), Note 2

Limits for the application of this method are not given in the National Annexes. The methods pursuant to clauses 10(4)...(7) of standard SFS-EN 1993-1-5 should be used.

Use of FEM calculations

C.2(1)

The FE-method to be used should be reliably verified. The user of the FE-method should have sufficient experience.

National Annex to standard SFS-EN 1993-1-6 Part 1-6: Strength and stability of shell structures

As regards standard SFS-EN 1993-1-6, the recommended values set forth in standard SFS-EN 1993-1-6 and all the annexes to standard SFS-EN 1993-1-6 are followed unless otherwise stated in this National Annex.

National choice is permitted in the following clauses of standard SFS-EN 1993-1-6:

- • 3.1(4)
- • 4.1.4(3)
- – 5.2.4(1)
- – 6.2.1(6), Note 2
- – 6.3(5)
- – 7.3.1(1) Note 2
- – 7.3.2(1)
- – 8.4.2(3)
- – 8.4.3(2)
- – 8.4.3(4), Note 1
- – 8.4.4(4)
- – 8.4.5(1)
- – 8.5.2(2)
- – 8.5.2(4), Note 1
- – 8.8.2(9)
- – 8.8.2(18)
- – 8.8.2(20), Note 1
- – 8.8.2(20), Note 2
- – 9.2.1(2)P
- – E.1.2.3(3), Note

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

Material properties

3.1(4)
When the temperature of steel is higher than 150°C, the mechanical properties for elevated temperatures that are used should be reliably determined.

LS4: Fatigue

4.1.4(3)
The recommended value should be used if no other rules are given in the application standards (for example, SFS-EN 1993-3- and SFS-EN 1993-4) or if there are no other reasons for using a lower value.

National Annex to standard SFS-EN 1993-1-7 Part 1-7: Plated structures subject to out of plane loading

As regards standard SFS-EN 1993-1-7, the recommended values set forth in standard SFS-EN 1993-1-7 and all the annexes to standard SFS-EN 1993-1-7 are followed unless otherwise stated in this National Annex.

National choice is permitted in the following clauses of standard SFS-EN 1993-1-7:
- 6.3.2(4), Note 1.

National Annex to standard SFS-EN 1993-1-8 Part 1-8: Design of joints

As regards standard SFS-EN 1993-1-8, the recommended values set forth in standard SFS-EN 1993-1-8 and all the annexes to standard SFS-EN 1993-1-8 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 1993-1-8:
- • 1.2.6
- • 2.2(2)
- • 3.1.1(3)
- • 3.4.2(1)
- – 5.2.1(2)
- – 6.2.7.2(9)

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

Reference Standards, Group 6: Rivets

1.2.6
Rivets pursuant to the following standards may be used:
- DIN 124 Halbrundniete – Nenndurchmesser 10 bis 36 mm (2011)
- DIN 302 Senkniete - Nenndurchmesser 10 bis 36 mm (2011)
- NF E25-726 - Fixations - Rivets pleins à tête ronde pour constructions métalliques (Fasteners - Structural round head plain rivets) (2011)
- SS 39, Nitar - Nitar med runt huvud för stålkonstruktioner - TypKN (1983)
- SS 318, Nitar - Nitar med sänkhuvud för stålkonstruktioner - Typ FN (1983).

Material pursuant to standard SFS-EN 10263-2 may be used in rivets:
- SFS-EN 10263-2 Steel rod, bars and wire for cold heading and cold extrusion. Part 2: Technical delivery conditions for steels not intended for heat treatment after cold working.

General requirements

2.2(2)
When calculating the resistance of welds, the prerequisite for the use of partial factor γ_{M2} = 1.25 is that the weld class is at least C according to standards SFS-EN ISO 5817. An “end crater pipe” (2025) must not exist along the effective length of the weld in any weld class.

General

3.1.1(3)
Only the use of bolt classes 8.8 and 10.9 is recommended.

Tension connections

3.4.2 (1)
The value for preload force is 0.7 $f_{ub}\,A_s$. In this case, bolted connections should be checked at a minimum in a similar manner as non-preloaded connections.

General

4.1
Standard SFS-EN 1993-1-8 does not provide instructions for suitable arc welding processes.

The design methods pursuant to standard SFS-EN 1993-1-8 concern the following arc welding processes:
111 – metal arc welding with covered electrode
114 – flux cored arc welding (without gas shielding)
12 – submerged arc welding
131 – MIG welding (inert shield gas)
135 – MAG welding (active shield gas)
136 – MAG - flux cored arc welding
137 – MIG - flux cored arc welding
141 – TIG welding (inert shield gas)

The reference numbers presented above are given in standard SFS-EN ISO 4063*: Welding and allied processes. Nomenclature of processes and reference numbers.*

Directional method

4.5.3.2
The value of β_w for steel grades pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3 should be determined based on yield strength as for steels pursuant to standard SFS-EN 10025.

Simplified method for design resistance of fillet weld

4.5.3.3

The value of β_w *for steel grades pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3 should be determined based on yield strength as for steels pursuant to standard SFS-EN 10025.*

Conditions for application

7.1.2

Shear rupture resistance for overlap type joints in structural hollow sections is checked according to the NCCI 1 document that follows this National Annex. See also clause 7.1.2(6) of National Annex to Standard SFS-EN 1993-1-8.

NCCI 1 for standard SFS-EN 1993-1-8: Overlap type joints in structural hollow sections

General

As regards overlap-type joints, the welding/non-welding of the hidden side of the brace member to thechord is presented in the execution specification.

When following the instructions presented in clauses (1) and (2), the nominal yield strength for hot rolled or cold formed structural hollow sections may be at most 460 N/mm^2, when the structural hollow section is treated as an end product. When the nominal yield strength of the end product is higher than 355 N/mm^2, the design resistances presented in clauses (1) and (2) are multiplied by 0.9.

According to the additional guidance presented in clause 7.1.2(6) of standard SFS-EN 1993-1-8, the following additional check is performed on the durability of overlap type joints:

When λov > λov.lim (circular, square or rectangular brace members) or when, for rectangular brace members, $h_i < b_i$ or $h_j < b_j$, the shearing of brace members from the chord in the direction of the chord is checked (see Figure 1).

In this case, the overlap limit value λov.lim is defined as follows:

- *$\lambda_{ov.lim}$ = 60%, when the hidden side of the lapped brace member is not welded to the chord*
- *$\lambda_{ov.lim}$ = 80%, when the hidden side of the lapped brace member is welded to the chord*

Standard SFS-EN 1993-1-8 does not provide instructions checking the shear rupture resistance in question. The following instructions presented in clauses (1) and (2) below are followed.

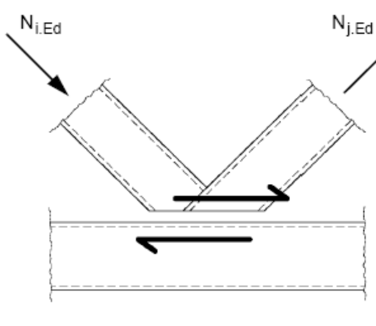


Figure 1. *Overlap type joint. Shearing of brace members from the chord.*

The instructions presented in Table 1 are followed depending on whether the chord is a structural hollow section or an I-profile.

Circular structural hollow sections as brace members

The shearing of the brace members is checked as follows:

When: *60% < λ_{ov} < 100%, when the hidden side of the lapped brace member is not welded to the chord*

or: *80% < λ_{ov} < 100%, when the hidden side of the lapped brace member is welded to the chord*

the design condition is checked as follows:

$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \le N_{s.Rd} \qquad (1.1)$$

where:

$$N_{s.Rd} = \frac{\pi}{4}\cdot\left[\frac{f_{ui}}{\sqrt{3}}\cdot\frac{\left[\left(\frac{100-\lambda_{ov}}{100}\right)\cdot 2d_i + d_{eff.i}\right]\cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}}\cdot\frac{\left(2d_j + c_s d_{eff.j}\right)\cdot t_j}{\sin\theta_j}\right]\cdot\frac{1}{\gamma_{M5}} \qquad (1.2)$$

When λ_{ov} = 100%, the design condition is checked as follows:

$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \le N_{s.Rd} \qquad (1.3)$$

where:

$$N_{s.Rd} = \frac{\pi}{4}\cdot\frac{f_{uj}}{\sqrt{3}}\cdot\frac{\left(3d_j + d_{eff.j}\right)\cdot t_j}{\sin\theta_j}\cdot\frac{1}{\gamma_{M5}} \qquad (1.4)$$

In expressions (1.1) – (1.4), subindex i refers to the lapping and subindex j refers to the lapped brace member in accordance with standard SFS-EN 1993-1-8.

The other symbols are defined as follows:

f_u *is the nominal yield strength for the brace member*

c_s *is the factor of the effective shear face area:*

$c_s = 1$, *when the hidden side of the lapped brace member is not welded to the chord*

$c_s = 2$, *when the hidden side of the lapped brace member is welded to the chord*

d_{eff} *is the effective diameter according to Table 1.*

All other symbols are pursuant to standard SFS-EN 1993-1-8.

Square or rectangular structural hollow sections as brace members

The shearing of the brace members is checked as follows:

When:	*60% < λ_{ov} < 100%, when the hidden side of the lapped brace member is not welded to the chord*
or:	*80% < λ_{ov} < 100%, when the hidden side of the lapped brace member is welded to the chord*
or:	*$h_i < b_i$ and $\lambda_{ov} < 100\%$*
or:	*$h_i < b_i$ and $\lambda_{ov} < 100\%$*

the design condition is checked as follows:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \tag{1.5}$$

where:

$$N_{s.Rd} = \left[\frac{f_{ui}}{\sqrt{3}} \cdot \frac{\left[\left(\frac{100 - \lambda_{ov}}{100} \right) \cdot 2h_i + b_{eff.i} \right] \cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}} \cdot \frac{\left(2h_j + c_s b_{eff.j} \right) \cdot t_j}{\sin\theta_j} \right] \cdot \frac{1}{\gamma_{M5}} \tag{1.6}$$

When λ_{ov} = 100%, the design condition is checked as follows:

$$N_{i.Ed} \cos\theta_i + N_{j.Ed} \cos\theta_j \leq N_{s.Rd} \tag{1.7}$$

where:

$$N_{s.Rd} = \frac{f_{uj}}{\sqrt{3}} \cdot \frac{\left(2h_j + b_j + b_{eff.j} \right) \cdot t_j}{\sin\theta_j} \cdot \frac{1}{\gamma_{M5}} \tag{1.8}$$

In expressions (1.5) – (1.8), subindex i refers to the lapping and subindex j refers to the lapped brace member in accordance with standard SFS-EN 1993-1-8.

The other symbols are defined as follows:

f_u	*is the nominal yield strength for the brace member*
c_s	*is the factor of the effective shear face area:* *c_s = 1, when the hidden side of the lapped brace member is not welded to the chord* *c_s = 2, when the hidden side of the lapped brace member is welded to the chord*
b_{eff}	*is the effective width pursuant to Table 1.*

All other symbols are pursuant to standard SFS-EN 1993-1-8.

Table 1. *Shearing of brace members in a lapped joint. Effective dimensions for brace members.*

		Brace members	
		Circular (CHS)	*Square or rectangular (RHS)*
Chord	*Circular (CHS)*	*Lapping CHS brace member in a CHS:* $d_{eff.i} = \frac{12}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot d_i$ $mutta \le d_i$	–
		Lapped CHS brace member in a CHS chord: $d_{eff.j} = \frac{12}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot d_j$ $mutta \le d_j$	–
	Square or rectangular (RHS)	*Lapping CHS brace member in an RHS chord:* $d_{eff.i} = \frac{10}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot d_i$ $mutta \le d_i$	*Lapping RHS brace member in an RHS chord:* $b_{eff.i} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i$ $mutta \le b_i$
		Lapped CHS brace member in an RHS chord: $d_{eff.j} = \frac{10}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot d_j$ $mutta \le d_j$	*Lapped RHS brace member in an RHS chord:* $b_{eff.j} = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yj} \cdot t_j} \cdot b_j$ $mutta \le b_j$
	I-profile	*Lapping CHS brace member in an I-chord:* $d_{eff.i} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yi}}$ $mutta \le d_i$	*Lapping RHS brace member in an I-chord:* $b_{eff.i} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yi}}$ $mutta \le b_i$
		Lapped CHS brace member in an I-chord: $d_{eff.j} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yj}}$ $mutta \le d_j$	*Lapped RHS brace member in an I-chord:* $b_{eff.j} = t_w + 2r + 7t_0 \cdot \frac{f_{y0}}{f_{yj}}$ $mutta \le b_j$

National Annex to standard SFS-EN 1993-1-9 Part 1-9: Fatigue

As regards standard SFS-EN 1993-1-9, the recommended values set forth in standard SFS-EN 1993-1-9 and all the annexes to standard SFS-EN 1993-1-9 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 1993-1-9:

– 1.1(2), Note 1
• 1.1(2), Note 2
– 2(2)
– 2(4)
• 3(2), Note 2
• 3(7)
• 5(2), Note 2
– 6.1(1)
– 6.2(2)
• 7.1(3), Note 2
– 7.1(5)
– 8(4), Note 2.

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

Scope

1.1(2), Note 2
Additional inspection requirements, see clause 3(2), note 2.

1.1(3)
Standard SFS-EN 1090-2 also allows cutting holes with an entirely thermal cutting process (e.g. plasma). The following instructions are also followed:

The instructions in standard SFS-EN 1993-1-9 do not apply if the holes are made with an entirely thermal cutting process (e.g. plasma).

In fatigue loaded structures, defects that reduce fatigue resistance are eliminated in a manner that ensures the design prerequisites pursuant to standard SFS-EN 1993-1-9 are met, or it is ensured that no defects larger than allowed in the design pursuant to standard SFS-EN 1993-1-9 occur.

Reliability considerations

3(2), Note 2

The following rules should be followed in the inspection programme:

a) Damage tolerant design should ensure that when damage occurs due to accidental action, deterioration of material, corrosion or fatigue, the remaining structure can sustain at least the used combination of actions without failure beyond an agreed extent, until the damage can be detected and the damaged structure can be repaired or replaced.

b) The combination of actions to be considered and the extent of failure to be accepted should be agreed between the client, the designer and the competent authority and recorded in the execution specification. When the damage tolerant concept is used, the execution specification should include information concerning the methods of inspection and inspection intervals to be used as well as the procedure to be followed when the structure has reached the end of its service life.

c) To ensure sufficient robustness, provisions should be made for inspection and maintenance at appropriate intervals that comply with the safety requirements. Guidance for the use, maintenance and inspection of the fatigue loaded structure should be given in the use and maintenance instructions of the fatigue loaded structure or of the building. Guidance for the use, maintenance and inspection of the fatigue loaded structure should be given to the owner of the structure during the final approval.

d) All structural parts of a fatigue loaded structure, including all connections, should be sufficiently accessible for appropriate inspection and maintenance. The real possibilities for carrying out the required inspections should be taken into account when the partial safety factors are chosen.

3(7)

Usually, the safe life concept is used.

Calculation of stresses

5(2), Note 2

Rules according to Standard SFS-EN 1993-1-5 should be used in the calculation of stresses for class 4 sections.

General

7.1(3), Note 2

When other fatigue strength categories are used in an individual application, the rules given in Note 1 are followed.

National Annex to standard SFS-EN 1993-1-10 Part 1-10: Material toughness and through-thickness properties

As regards standard SFS-EN 1993-1-10, the recommended values set forth in standard SFS-EN 1993-1-10 and all the annexes to standard SFS-EN 1993-1-10 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 1993-1-10:
- 2.2(5), Note 1
- 2.2(5), Note 3
- 2.2(5), Note 4
- 3.1(1).

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

General

2.1(2)
Requirements for fracture toughness for elements only in compression are set according to the recommendations of clause 3.2.3(3)B in standard SFS-EN 1993-1-1, see National Annex to standard SFS-EN 1993-1-1.

Method

2.2(5)
Unless the correction term is specified in more detail, the selected value for the correction term ΔT_r that accounts for radiation loss is $\Delta T_r = -5°C$.

2.2(5), Note 1
Other reliability requirements are not given. The recommended value ΔTR = 0°C should be used.

2.2(5), Note 3
Table 2.1 should be applied as such.

2.2(5), Note 4
Table 2.1 of standard SFS-EN 1993-1-10 may be applied up to steel grade S690. Values for steel grade S700 are given in standard SFS-EN 1993-1-12. Values for some other steel grades are also given in standard SFS-EN 1993-1-12. For steels pursuant to standard SFS-EN 10149-2 and standard SFS-EN 10149-3, Table 1 should be used. See also clause 3.1(2) of National Annex to standard SFS-EN 1993-1-1.

Table 1. Maximum permissible values of element thickness *t* [mm] for steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3.

Steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3

Steel grade	Subgrade	Charpy energy KV		Reference temperature T_{Ed} [°C]													
		T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	-60	-70	-80	-90	-100	-110	-120
				$\sigma_{Ed} = 0.75 \cdot f_y(t)$													
S260	NC	-20	40	138	116	97	81	67	56	46	39	32	27	23	19	16	14
S315	MC NC	-20	40	120	100	83	69	57	47	39	32	27	22	18	15	13	11
S355	MC NC	-20	40	109	91	75	62	51	42	35	28	23	19	16	13	11	9
S420	MC NC	-20	40	95	79	65	53	44	36	29	24	19	16	13	11	9	7
S460	MC	-20	40	88	73	60	49	40	32	26	21	17	14	12	9	8	6

Note 1: For these steel grades, the requirement for fracture toughness is determined as follows:

SFS-EN 10149-1: Clause 11: Option 5

Note 2: Thickness range according to standard SFS-EN 10149: max. 20 mm

Table 1. (continued) Maximum permissible values of element thickness *t* [mm] for steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3.

Steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3																	
Steel grade	Subgrade	Charpy energy KV		Reference temperature T_{Ed} [°C]													
				10	0	-10	-20	-30	-40	-50	-60	-70	-80	-90	-100	-110	-120
		T [°C]	J_{min}	$\sigma_{Ed} = 0.50 \cdot f_y(t)$													
S260	NC	-20	40	184	159	136	116	99	84	71	60	51	44	37	32	28	25
S315	MC NC	-20	40	166	142	121	103	87	73	62	52	44	37	32	27	24	21
S355	MC NC	-20	40	155	132	112	95	80	67	56	47	40	34	28	24	21	18
S420	MC NC	-20	40	139	118	99	83	70	58	49	41	34	29	24	20	17	15
S460	MC	-20	40	130	110	93	78	65	54	45	37	31	26	22	18	16	14

Note 1: For these steel grades, the requirement for fracture toughness is determined as follows:

SFS-EN 10149-1: Clause 11: Option 5

Note 2: Thickness range according to standard SFS-EN 10149: max. 20 mm

Table 1. (continued) Maximum permissible values of element thickness t [mm] for steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3.

Steels pursuant to standards SFS-EN 10149-2 and SFS-EN 10149-3																	
Steel grade	Subgrade	Charpy energy KV		Reference temperature T_{Ed} [°C]													
		T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	-60	-70	-80	-90	-100	-110	-120
				$\sigma_{Ed} = 0.25 \cdot f_y(t)$													
S260	NC	-20	40	199	199	193	169	147	128	111	96	84	73	64	56	50	44
S315	MC NC	-20	40	199	199	180	157	136	118	102	88	76	66	58	51	45	40
S355	MC NC	-20	40	199	197	172	149	129	111	96	83	71	62	54	47	41	37
S420	MC NC	-20	40	199	183	159	138	118	102	87	75	64	55	48	42	37	33
S460	MC	-20	40	199	176	152	131	113	96	82	71	61	52	45	39	34	30
Note 1: For these steel grades, the requirement for fracture toughness is determined as follows: SFS-EN 10149-1: Clause 11: Option 5 **Note 2:** Thickness range according to standard SFS-EN 10149: max. 20 mm																	

Maximum allowed material thicknesses

2.3.1

In order to account for cold-forming and to prevent brittle failure, the instructions in the NCCI 1 document appended to this National Annex are followed.

Selection of material based on through-thickness properties

3.1(1)

Class 1 should be used.

NCCI 1 for standard SFS-EN 1993-1-10: Determining temperature conversion

Expression (2.4) in clause 2.3.1(2) of standard SFS-EN 1993-1-10 is modified as follows:

$\Delta T_{\varepsilon.cf} = -0\left[{}^{0}C\right]$, *when* $\varepsilon_{cf} \leq 2\%$ *(1.1)*

$\Delta T_{\varepsilon.cf} = -3\varepsilon_{cf}\left[{}^{0}C\right]$ *,when* ε_{cf}*>2%, however* $\Delta T_{\varepsilon.cf} \geq -45\left[{}^{0}C\right]$ *(1.2)*

The symbol $\Delta T_{\varepsilon_{cf}}$ *in standard SFS-EN 1993-1-10 corresponds to the symbol* $\Delta T_{\varepsilon.cf}$ *used here.*

Instead of the degree of cold-forming ε_{cf} *, the calculations use equivalent effective elongation* ε_{eff} *, where* ε_{eff} *refers to the Charpy-V average of the test piece's net cross section average elastic elongation in the longitudinal direction of the curved wall. The value depends on the wall thickness and the interior bending radius* r_i *according to Table 1.*

Table 1 presents the determination of the maximum elastic elongation for a cold-formed sheet part ε_{pl} *and the effective elongation* ε_{eff} *.*

For circular structural hollow sections pursuant to standard SFS-EN 10219, the temperature conversions are as follows:

$\Delta T_{\varepsilon.cf} = -0\left[{}^{0}C\right]$, *when* $\frac{r_i}{t} > 15$ *(1.3)*

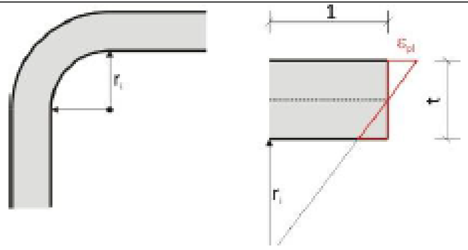
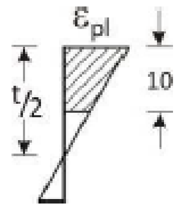
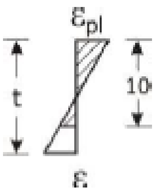
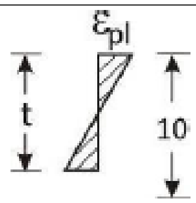
$\Delta T_{\varepsilon.cf} = -20\left[{}^{0}C\right]$, *when* $\frac{r_i}{t} \leq 15$ *(1.4)*

For rectangular structural hollow sections pursuant to standard SFS-EN 10219, the temperature conversions are as follows:

$\Delta T_{\varepsilon.cf} = -35\left[{}^{o}C\right]$, *when t* $\leq$ *16 mm* *(1.5)*

$\Delta T_{\varepsilon.cf} = -45\left[{}^{0}C\right]$, *when 16 mm < t* $\leq$ *40 mm* *(1.6)*

Table 1. Determining the variables ε_{pl} and ε_{eff} for cold-formed areas

Determining maximum elastic elongation ε_{pl}		
		$\varepsilon_{pl} = \dfrac{t}{2r_i + t}$
Determining effective elongation ε_{eff}		
t (mm)	*Variable* ε_{pl} *distribution*	$\varepsilon_{\mathrm{eff}}$
≥ 20		$\varepsilon_{pl}(1-\dfrac{10}{t})$
< 20 *≥ 10*		$\dfrac{\varepsilon_{pl}}{2}(\dfrac{t}{20}+\dfrac{(20-t)^2}{20t})$
< 10		$\dfrac{\varepsilon_{pl}}{2}\dfrac{t}{10}$

Determining maximum allowed structural member thickness

Table 2 presents the maximum allowed structural member thickness when the reference temperature is in the range –60°C …–120°C.

Table 2. Maximum allowed structural member thickness t [mm]

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.75 \cdot f_y(t)$						
S235	*JR*	*+20*	*27*	*18*	*16*	*14*	*12*	*11*	*10*	*9*
	J0	*0*	*27*	*25*	*21*	*18*	*16*	*14*	*12*	*11*
	J2	*-20*	*27*	*36*	*30*	*25*	*21*	*18*	*16*	*14*
S275	*JR*	*+20*	*27*	*15*	*13*	*11*	*10*	*9*	*8*	*7*
	J0	*0*	*27*	*22*	*18*	*15*	*13*	*11*	*10*	*9*
	J2	*-20*	*27*	*31*	*26*	*22*	*18*	*15*	*13*	*11*
	M, N	*-20*	*40*	*37*	*31*	*25*	*21*	*18*	*15*	*13*
	ML, NL	*-50*	*27*	*54*	*45*	*37*	*31*	*26*	*22*	*18*
S355	*JR*	*+20*	*27*	*11*	*10*	*8*	*7*	*6*	*5*	*5*
	J0	*0*	*27*	*16*	*14*	*11*	*10*	*8*	*7*	*6*
	J2	*-20*	*27*	*24*	*20*	*16*	*14*	*11*	*10*	*8*
	K2, M, N	*-20*	*40*	*28*	*23*	*19*	*16*	*13*	*11*	*9*
	ML, NL	*-50*	*27*	*43*	*35*	*29*	*24*	*20*	*16*	*14*
S420	*M, N*	*-20*	*40*	*24*	*19*	*16*	*13*	*11*	*9*	*7*
	ML, NL	*-50*	*27*	*36*	*29*	*24*	*20*	*16*	*13*	*11*
S460	*Q*	*-20*	*30*	*18*	*15*	*12*	*10*	*8*	*7*	*6*
	M, N	*-20*	*40*	*21*	*17*	*14*	*12*	*9*	*8*	*6*
	QL	*-40*	*30*	*28*	*23*	*18*	*15*	*12*	*10*	*8*
	ML, NL	*-50*	*27*	*33*	*27*	*22*	*18*	*14*	*12*	*10*
	QL1	*-60*	*30*	*42*	*34*	*28*	*23*	*18*	*15*	*12*
S690	*Q*	*0*	*40*	*8*	*6*	*5*	*3*	*2*	*-*	*-*
	Q	*-20*	*30*	*11*	*8*	*7*	*5*	*4*	*2*	*1*
	QL	*-20*	*40*	*13*	*10*	*8*	*6*	*5*	*3*	*2*
	QL	*-40*	*30*	*17*	*13*	*11*	*8*	*7*	*5*	*4*
	QL1	*-40*	*40*	*20*	*16*	*13*	*10*	*8*	*6*	*5*
	QL1	*-60*	*30*	*27*	*21*	*17*	*13*	*11*	*8*	*7*

Table 2. (continued) Maximum allowed structural member thickness t [mm]

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.50 \cdot f_y(t)$						
S235	*JR*	*+20*	*27*	*31*	*27*	*24*	*21*	*19*	*18*	*16*
	J0	*0*	*27*	*41*	*35*	*31*	*27*	*24*	*21*	*19*
	J2	*-20*	*27*	*55*	*47*	*41*	*35*	*31*	*27*	*24*
S275	*JR*	*+20*	*27*	*27*	*24*	*21*	*18*	*17*	*15*	*14*
	J0	*0*	*27*	*36*	*31*	*27*	*24*	*21*	*18*	*17*
	J2	*-20*	*27*	*49*	*42*	*36*	*31*	*27*	*24*	*21*
	M, N	*-20*	*40*	*58*	*49*	*42*	*36*	*31*	*27*	*23*
	ML, NL	*-50*	*27*	*81*	*69*	*58*	*49*	*42*	*36*	*31*
S355	*JR*	*+20*	*27*	*21*	*18*	*16*	*14*	*13*	*11*	*10*
	J0	*0*	*27*	*29*	*25*	*21*	*18*	*16*	*14*	*13*
	J2	*-20*	*27*	*40*	*34*	*29*	*25*	*21*	*18*	*16*
	K2, M, N	*-20*	*40*	*47*	*40*	*34*	*28*	*24*	*21*	*18*
	ML, NL	*-50*	*27*	*68*	*57*	*48*	*40*	*34*	*29*	*25*
S420	*M, N*	*-20*	*40*	*41*	*34*	*29*	*24*	*20*	*17*	*15*
	ML, NL	*-50*	*27*	*59*	*49*	*41*	*34*	*29*	*24*	*21*
S460	*Q*	*-20*	*30*	*33*	*27*	*23*	*19*	*16*	*14*	*12*
	M, N	*-20*	*40*	*37*	*31*	*26*	*22*	*18*	*16*	*14*
	QL	*-40*	*30*	*47*	*39*	*33*	*27*	*23*	*19*	*16*
	ML, NL	*-50*	*27*	*54*	*45*	*38*	*31*	*26*	*22*	*19*
	QL1	*-60*	*30*	*68*	*57*	*47*	*39*	*33*	*27*	*23*
S690	*Q*	*0*	*40*	*16*	*13*	*11*	*9*	*8*	*6*	*5*
	Q	*-20*	*30*	*21*	*17*	*14*	*12*	*10*	*8*	*7*
	QL	*-20*	*40*	*24*	*20*	*16*	*13*	*11*	*9*	*8*
	QL	*-40*	*30*	*31*	*26*	*21*	*17*	*14*	*12*	*10*
	QL1	*-40*	*40*	*36*	*30*	*24*	*20*	*16*	*13*	*11*
	QL1	*-60*	*30*	*47*	*38*	*31*	*26*	*21*	*17*	*14*

Table 2. (continued) Maximum allowed structural member thickness t [mm]

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.25 \cdot f_y(t)$						
S235	*JR*	*+20*	*27*	*53*	*47*	*42*	*39*	*35*	*33*	*31*
	J0	*0*	*27*	*67*	*59*	*53*	*47*	*42*	*39*	*35*
	J2	*-20*	*27*	*88*	*77*	*67*	*59*	*53*	*47*	*42*
S275	*JR*	*+20*	*27*	*49*	*43*	*39*	*35*	*32*	*30*	*28*
	J0	*0*	*27*	*62*	*55*	*49*	*43*	*39*	*35*	*32*
	J2	*-20*	*27*	*82*	*71*	*62*	*55*	*49*	*43*	*39*
	M, N	*-20*	*40*	*94*	*81*	*71*	*62*	*55*	*48*	*43*
	ML, NL	*-50*	*27*	*126*	*109*	*95*	*82*	*71*	*62*	*55*
S355	*JR*	*+20*	*27*	*42*	*37*	*33*	*30*	*27*	*25*	*23*
	J0	*0*	*27*	*54*	*47*	*42*	*37*	*33*	*30*	*27*
	J2	*-20*	*27*	*72*	*62*	*54*	*47*	*42*	*37*	*33*
	K2, M, N	*-20*	*40*	*83*	*71*	*62*	*54*	*47*	*41*	*37*
	ML, NL	*-50*	*27*	*112*	*97*	*83*	*72*	*62*	*54*	*47*
S420	*M, N*	*-20*	*40*	*75*	*64*	*55*	*48*	*42*	*37*	*33*
	ML, NL	*-50*	*27*	*103*	*88*	*75*	*65*	*56*	*48*	*42*
S460	*Q*	*-20*	*30*	*63*	*54*	*47*	*41*	*36*	*31*	*28*
	M, N	*-20*	*40*	*71*	*61*	*52*	*45*	*39*	*34*	*30*
	QL	*-40*	*30*	*86*	*74*	*63*	*54*	*47*	*41*	*36*
	ML, NL	*-50*	*27*	*97*	*83*	*71*	*61*	*53*	*45*	*39*
	QL1	*-60*	*30*	*118*	*101*	*86*	*74*	*63*	*54*	*47*
S690	*Q*	*0*	*40*	*38*	*32*	*28*	*24*	*21*	*18*	*16*
	Q	*-20*	*30*	*46*	*39*	*34*	*29*	*25*	*22*	*19*
	QL	*-20*	*40*	*52*	*44*	*38*	*32*	*28*	*24*	*21*
	QL	*-40*	*30*	*65*	*55*	*46*	*39*	*34*	*29*	*25*
	QL1	*-40*	*40*	*73*	*62*	*52*	*44*	*38*	*32*	*28*
	QL1	*-60*	*30*	*91*	*77*	*65*	*55*	*46*	*39*	*34*

National Annex to standard SFS-EN 1993-1-11 Part 1-11: Design of structures with tension components

As regards standard SFS-EN 1993-1-11, the recommended values set forth in standard SFS-EN 1993-1-11 and all the annexes to standard SFS-EN 1993-1-11 are followed unless otherwise stated in this National Annex.

National choice is permitted in the following clauses of standard SFS-EN 1993-1-11:

– 2.3.6(1)
• 2.3.6(2), Note 1
– 2.4.1(1)
– 3.1(1), Note 6
• 4.4(2), Note 1
• 4.5(4), Note 1
– 5.2(3)
– 5.3(2)
– 6.2(2), Note 4
– 6.3.2(1)
– 6.3.4(1)
– 6.4.1(1)P, Note 1
– 7.2(2), Note 1
• A.4.5.1(1)
• A.4.5.2(1)
• B(6).

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

Replacement and loss of tension components

2.3.6(2), Note 1
Sudden loss of any one tension component should always be taken into account as a consequence of impact load and design should be made as accidental design situation. Other situations should be determined case by case in each project.

Corrosion protection of the exterior of group B tension components

4.4(2), Note 1
The steel grade of the stainless steel should be selected according to Annex A to standard SFS-EN 1993-1-4+A1:2015.

Corrosion protection of group C tension components

4.5(4), Note 1
Acceptable corrosion protection fillers in cables of building structures are grease, wax, soft resin and cement grout. The use of cement grout is not accepted in fatigue loaded structures and in structures which are designed in a such a way that individual wires are replaceable during the planned service life of the structure. The filler should work acceptably at the service temperature.

Water-proofing

A.4.5.1(1)
Testing instructions concerning water-proofing should be presented for each project.

Corrosion protection barriers

A.4.5.2
Testing instructions concerning corrosion protection barriers should be presented for each project.

Transport, storage and handling

B(6)
Instructions for monitoring and inspections should be presented for each project.

National Annex to standard SFS-EN 1993-1-12 Part 1-12: Extension of EN 1993 up to steel grades S700

As regards standard SFS-EN 1993-1-12, the recommended values set forth in standard SFS-EN 1993-1-12 and all the annexes to standard SFS-EN 1993-1-12 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 1993-1-12:

- • 2.1 (3.1(2))
- – 2.1 (3.2.2(1))
- – 2.1 (5.4.3(1))
- • 2.1 (6.2.3(2))
- – 2.8 (4.2(2))
- – 3(1).

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

Additional provisions for standard EN 1993-1-1

2.1 (3.1(2))

Steel grades given in Table 1 and 2 of standard EN 1993-1-12 and their recommended values should be used. In addition, such steel grades may be used whose properties have been reliably analysed and whose analysis references clause 2.1 (3.1(2)) of the National Annex to standard SFS-EN 1993-1-12, and it is stated that this steel grade may be used according to standard SFS-EN 1993-1-12 and its National Annex.

2.1 (6.2.3(2))

A value of $\gamma_{M12} = \frac{f_u}{f_y}\gamma_{M0}$ should be used, where γ_{M0} is determined according to the National Annex to standard SFS-EN 1993-1-1.

Additional provisions for standard EN 1993-1-8

2.8 (1.1(1))

Standard SFS-EN 1993-1-12 does not present instructions concerning the effects of welding on the properties of the parent metal in the area influenced by the welding heat (HAZ) immediately next to the weld. The following instructions are followed.

Additional instructions for steel grades with a strength above S460 but at most S700

The yield strength of the parent metal f_y *is multiplied by* k_{HAZ} *immediately next to the weld (within the area influenced by the welding heat, HAZ) as follows, unless other values are experimentally determined to be more correct:*

k_{HAZ} *= 1*	*when* $f_y \leq 500$ *N/mm2*	*(1.1)*
$k_{HAZ} =$ *= 0.85*	*when* $f_y = 700$ *N/mm2*	*(1.2)*

Intermediate values are interpolated linearly.

The above does not apply to clause 2.8/7.1.1(4) of standard SFS-EN 1993-1-12.

Additional provisions for standard EN 1993-1-10

2.10 (2.3.2(1))
Determining maximum allowed structural member thickness

Table 1 presents the maximum allowed structural member thickness for steel grades S500, S550, S600, S650 and S700 pursuant to standard SFS-EN 10149-2 when the reference temperature is within the range –60°C …–120°C.

Table 2 presents the maximum allowed structural member thickness for steel grades S500, S550 and S620 pursuant to standard SFS-EN 10025-6 when the reference temperature is within the range –60°C …–120°C.

Table 1. Maximum allowed structural member thickness t [mm] for steel grades S500, S550, S600, S650 and S700 pursuant to SFS-EN 10149-2 when the reference temperature is within the range –60°C…–120°C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.75 \cdot f_y(t)$						
S500	*MC*	*-20*	*40*	*19*	*16*	*13*	*10*	*8*	*7*	*5*
S550	*MC*	*-20*	*40*	*17*	*14*	*11*	*9*	*7*	*6*	*5*
S600	*MC*	*-20*	*40*	*15*	*12*	*10*	*8*	*6*	*5*	*4*
S650	*MC*	*-20*	*40*	*14*	*11*	*9*	*7*	*5*	*4*	*2*
S700	*MC*	*-20*	*40*	*12*	*10*	*8*	*6*	*4*	*3*	*1*

Note 1: For these steel grades, the requirement for fracture toughness is determined as follows: SFS-EN 10149-1: Clause 11: Option 5

Note 2: Thickness range according to standard SFS-EN 10149: max. 20 mm

Table 1. (continued) Maximum allowed structural member thickness t [mm] for steel grades S500, S550, S600, S650 and S700 pursuant to SFS-EN 10149-2 when the reference temperature is within the range –60°C…–120°C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
				-60	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
		T [°C]	*J_{min}*	*$\sigma_{Ed} = 0.50 \cdot f_y(t)$*						
S500	*MC*	*-20*	*40*	*34*	*29*	*24*	*20*	*17*	*14*	*12*
S550	*MC*	*-20*	*40*	*31*	*26*	*21*	*18*	*15*	*13*	*11*
S600	*MC*	*-20*	*40*	*28*	*23*	*19*	*16*	*13*	*11*	*9*
S650	*MC*	*-20*	*40*	*26*	*21*	*18*	*14*	*12*	*10*	*8*
S700	*MC*	*-20*	*40*	*24*	*19*	*16*	*13*	*11*	*9*	*7*

Note 1: For these steel grades, the requirement for fracture toughness is determined as follows: SFS-EN 10149-1: Clause 11: Option 5

Note 2: Thickness range according to standard SFS-EN 10149: max. 20 mm

Table 1. (continued) Maximum allowed structural element thickness t [mm] for steel grades S500, S550, S600, S650 and S700 pursuant to SFS-EN 10149-2 when the reference temperature is within the range –60°C…-120 °C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
				-60	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
		T [°C]	*J_{min}*	*$\sigma_{Ed} = 0.25 \cdot f_y(t)$*						
S500	*MC*	*-20*	*40*	*67*	*57*	*49*	*42*	*37*	*32*	*28*
S550	*MC*	*-20*	*40*	*62*	*53*	*46*	*39*	*34*	*30*	*26*
S600	*MC*	*-20*	*40*	*58*	*50*	*42*	*36*	*31*	*27*	*24*
S650	*MC*	*-20*	*40*	*55*	*47*	*40*	*34*	*29*	*25*	*22*
S700	*MC*	*-20*	*40*	*52*	*44*	*37*	*32*	*27*	*24*	*21*

Note 1: For these steel grades, the requirement for fracture toughness is determined as follows: SFS-EN 10149-1: Clause 11: Option 5

Note 2: Thickness range according to standard SFS-EN 10149: max. 20 mm

Table 2. Maximum allowed structural element thickness t [mm] for steel grades S500, S550 and S620 pursuant to SFS-EN 10025-6 when the reference temperature is within the range –60°C...–120°C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	J_{min}	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.75 \cdot f_y(t)$						
S500	*Q*	*0*	*40*	*13*	*10*	*8*	*7*	*5*	*4*	*4*
	Q	*-20*	*30*	*17*	*13*	*11*	*9*	*7*	*6*	*5*
	QL	*-20*	*40*	*19*	*16*	*13*	*10*	*8*	*7*	*5*
	QL	*-40*	*30*	*25*	*21*	*17*	*13*	*11*	*9*	*7*
	QL1	*-40*	*40*	*29*	*24*	*19*	*16*	*13*	*10*	*8*
	QL1	*-60*	*30*	*39*	*31*	*25*	*21*	*17*	*13*	*11*
S550	*Q*	*0*	*40*	*11*	*9*	*7*	*6*	*5*	*4*	*2*
	Q	*-20*	*30*	*15*	*12*	*9*	*8*	*6*	*5*	*4*
	QL	*-20*	*40*	*17*	*14*	*11*	*9*	*7*	*6*	*5*
	QL	*-40*	*30*	*23*	*18*	*15*	*12*	*9*	*8*	*6*
	QL1	*-40*	*40*	*26*	*21*	*17*	*14*	*11*	*9*	*7*
	QL1	*-60*	*30*	*35*	*28*	*23*	*18*	*15*	*12*	*9*
S620	*Q*	*0*	*40*	*9*	*7*	*6*	*4*	*3*	*2*	*1*
	Q	*-20*	*30*	*12*	*10*	*8*	*6*	*5*	*4*	*2*
	QL	*-20*	*40*	*15*	*12*	*9*	*7*	*6*	*4*	*3*
	QL	*-40*	*30*	*20*	*16*	*12*	*10*	*8*	*6*	*5*
	QL1	*-40*	*40*	*23*	*18*	*15*	*12*	*9*	*7*	*6*
	QL1	*-60*	*30*	*30*	*24*	*20*	*16*	*12*	*10*	*8*

Table 2.(continued) Maximum allowed structural element thickness t [mm] for steel grades S500, S550 and S620 pursuant to SFS-EN 10025-6 when the reference temperature is within the range –60°C…–120°C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.50 \cdot f_y(t)$						
S500	*Q*	*0*	*40*	*24*	*20*	*17*	*14*	*12*	*10*	*9*
	Q	*-20*	*30*	*30*	*25*	*21*	*18*	*15*	*13*	*11*
	QL	*-20*	*40*	*34*	*29*	*24*	*20*	*17*	*14*	*12*
	QL	*-40*	*30*	*44*	*36*	*30*	*25*	*21*	*18*	*15*
	QL1	*-40*	*40*	*50*	*41*	*34*	*29*	*24*	*20*	*17*
	QL1	*-60*	*30*	*63*	*53*	*44*	*36*	*30*	*25*	*21*
S550	*Q*	*0*	*40*	*21*	*18*	*15*	*13*	*11*	*9*	*8*
	Q	*-20*	*30*	*27*	*23*	*19*	*16*	*13*	*11*	*10*
	QL	*-20*	*40*	*31*	*26*	*21*	*18*	*15*	*13*	*11*
	QL	*-40*	*30*	*40*	*33*	*27*	*23*	*19*	*16*	*13*
	QL1	*-40*	*40*	*46*	*38*	*31*	*26*	*21*	*18*	*15*
	QL1	*-60*	*30*	*58*	*48*	*40*	*33*	*27*	*23*	*19*
S620	*Q*	*0*	*40*	*19*	*15*	*13*	*11*	*9*	*8*	*7*
	Q	*-20*	*30*	*24*	*20*	*16*	*14*	*11*	*9*	*8*
	QL	*-20*	*40*	*27*	*22*	*19*	*15*	*13*	*11*	*9*
	QL	*-40*	*30*	*35*	*29*	*24*	*20*	*16*	*14*	*11*
	QL1	*-40*	*40*	*40*	*33*	*27*	*22*	*19*	*15*	*13*
	QL1	*-60*	*30*	*52*	*43*	*35*	*29*	*24*	*20*	*16*

Table 2. (continued) Maximum allowed structural element thickness t [mm] for steel grades S500, S550 and S620 pursuant to SFS-EN 10025-6 when the reference temperature is within the range –60°C…–120°C.

Steel grade	*Subgrade*	*Charpy energy KV*		*Reference temperature T_{Ed} [°C]*						
		T [°C]	*J_{min}*	*-60*	*-70*	*-80*	*-90*	*-100*	*-110*	*-120*
				$\sigma_{Ed} = 0.25 \cdot f_y(t)$						
S500	*Q*	*0*	*40*	*49*	*42*	*37*	*32*	*28*	*25*	*23*
	Q	*-20*	*30*	*60*	*51*	*44*	*38*	*33*	*29*	*26*
	QL	*-20*	*40*	*67*	*57*	*49*	*42*	*37*	*32*	*28*
	QL	*-40*	*30*	*82*	*70*	*60*	*51*	*44*	*38*	*33*
	QL1	*-40*	*40*	*92*	*78*	*67*	*57*	*49*	*42*	*37*
	QL1	*-60*	*30*	*112*	*96*	*82*	*70*	*60*	*51*	*44*
S550	*Q*	*0*	*40*	*46*	*39*	*34*	*30*	*26*	*23*	*21*
	Q	*-20*	*30*	*56*	*48*	*41*	*35*	*31*	*27*	*24*
	QL	*-20*	*40*	*62*	*53*	*46*	*39*	*34*	*30*	*26*
	QL	*-40*	*30*	*77*	*65*	*56*	*48*	*41*	*35*	*31*
	QL1	*-40*	*40*	*86*	*73*	*62*	*53*	*46*	*39*	*34*
	QL1	*-60*	*30*	*106*	*90*	*77*	*65*	*56*	*48*	*41*
S620	*Q*	*0*	*40*	*41*	*35*	*31*	*27*	*23*	*21*	*18*
	Q	*-20*	*30*	*51*	*43*	*37*	*32*	*28*	*24*	*21*
	QL	*-20*	*40*	*57*	*48*	*41*	*35*	*31*	*27*	*23*
	QL	*-40*	*30*	*70*	*60*	*51*	*43*	*37*	*32*	*28*
	QL1	*-40*	*40*	*79*	*67*	*57*	*48*	*41*	*35*	*31*
	QL1	*-60*	*30*	*98*	*83*	*70*	*60*	*51*	*43*	*37*

National Annex to standard SFS-EN 1993-3-1 Part 3-1: Towers, masts and chimneys. Towers and masts

As regards standard SFS-EN 1993-3-1, the recommended values set forth in standard SFS-EN 1993-3-1 and all the annexes to standard SFS-EN 1993-3-1 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 1993-3-1:

- • 2.1.1(3)P
- • 2.3.1(1)
- • 2.3.2(1)
- • 2.3.6(2), Note 2
- – 2.3.7(1)
- • 2.3.7(4)
- – 2.5(1)
- • 2.6(1)
- • 4.1(1), Note 1
- • 4.2(1)
- – 5.1(6)
- – 5.2.4(1)
- • 6.1(1), Note 1
- – 6.3.1(1), Note 2
- – 6.4.1(1)
- – 6.4.2(2)
- – 6.5.1(1)
- • 7.1(1)
- • 9.5(1)
- • A.1(1)
- – A.2(1)P, Note 2
- • A.2(1)P, Note 3
- – B.1.1(1)
- – B.2.1.1(5)
- – B.2.3(1), Note 4
- – B.2.3(3)
- – B.3.2.2.6(4), Note 1
- – B.3.3(1)
- – B.3.3(2)
- – B.4.3.2.2(2), Note 2
- – B.4.3.2.3(1), Note 2
- – B.4.3.2.8.1(4), Note 2

- C.2(1)
- C.6.(1)
- – D.1.1(2)
- – D.1.2(2)
- D.3(6), Note 1
- – D.3(6), Note 2
- – D.4.1(1)
- D.4.2(3)
- D.4.3(1)
- – D.4.4(1)
- – F.4.2.1(1)
- – F.4.2.2(2)
- – G.1(3)
- – H.2(5)
- – H.2(7), Note 2.

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

Basic requirements

2.1.1(3)P
Guys breakdown will be taken into account only for consequences class 3 masts guyed from two or more planes.

Wind actions

2.3.1(1)
According to its scope, standard SFS-EN 1991-1-4 is not to be used to determine wind actions on masts. The determination of wind actions on masts should be based on the following rules:

a) The unmodified basic wind velocity value covering the entire country is $v_{b,0}$ = 21 m/s.

b) The terrain categories are pursuant to standard SFS-EN 1991-1-4, with the following specifications and additions:

Type 0:	Open sea, outer archipelago and open coastal areas
Type 0+:	Scattered inner archipelago and sheltered coastal areas
Type I:	Tight inner archipelago, large lake areas and wide fields
Type II+:	Variable inland terrain (forests, forest clearings, fields, lakes, individual buildings or groups of buildings)

Class II is used in fell areas unless specified otherwise in the execution specification.

c) The values from standard SFS-EN 1991-1-4 are used for terrain constants z_0 and k_r with the following exceptions:
   Type 0: $z_0 = 0.003$ and $k_r = 0.180$
   Type 0+: $z_0 = 0.003$ and $k_r = 0.167$
   Type II+: $z_0 = 0.095$ and $k_r = 0.195$

Wind velocities and pressures are calculated according to standard SFS-EN 1991-1-4. Since the equations are valid only to a height of 200 m, constant values calculated for the level of 200 m should be used when determining wind loads for mast sections above a height of 200 m.

The temperature used for wind actions without ice is –20°C.

When designing masts of over 100 metres in height on fells, the effect of so-called temperature inversion must also be taken into account with a separate examination where the velocity relating to the phenomenon is constant across the entire mast (peak velocity coefficient = 1.00).

Ice loads

2.3.2(1)
The rules in Annex C should be followed. The values of the ice loads and the combinations of wind and ice with the associated combination factors are given in Annex C of this National Annex.

Imposed loads

2.3.6(2), Note 1
In slender structures, where the loads caused by persons may affect the design of the structural components, the structure should be checked for the following load combination in erection or maintenance situations, taking into account the following actions:
- Reduced wind load (no ice), temperature 0°C
- Person in the mast (at an unfavourable location), characteristic weight 1 kN, effective wind area 1.0 m^2
- Equivalent horizontal characteristic force from the movements of a person 0.5 kN
- Other simultaneous erection/maintenance loads (hoist etc.)

In the calculation of the wind pressure, the parameters of terrain type should be applied in flat areas, independently of the real type or shape of the terrain at the site. The combination of actions is represented in the following expression:

$$\gamma_G \cdot G_k + \gamma_E \cdot Q_{k,E} + \gamma_W \cdot \psi_W \cdot Q_{k,w} \qquad (1.1)$$

where:

G_k is the characteristic value for self-weights of the structure and fixed equipment

$Q_{k,E}$ is the characteristic value for loads caused by erection work, persons etc.

$Q_{k,w}$ is the characteristic value for wind actions (including wind loads due to persons)

γ_G is the partial factor for self-weight, $\gamma_G = 1.15$

γ_E is the partial factor for erection actions, $\gamma_E = 1.5$

γ_W is the partial factor for wind actions, $\gamma_W = 1.5$

ψ_W is the combination factor for wind actions, $\psi_W = 0.5$

Other actions

2.3.7(4)

The loads due to the erection of a mast shall be taken into account in the design (i.e. erection by a derrick or a crane, pre-tensioning of guys etc.).

In the erection of a guyed mast, the case where any span between two adjacent guy levels is installed but where the guys at the upper guy level are not yet installed should be checked. The combination of actions is represented in the following expression:

$$\gamma_G \cdot G_k + \gamma_W \cdot \psi_W \cdot Q_{k,w} \quad (1.2)$$

where:

G_k is the characteristic value for self-weights of the structure and equipment,

$Q_{k,w}$ is the characteristic value for wind actions,

γ_G is the partial factor for self-weight, Table 2

γ_W is the partial factor for wind actions, Table 2

ψ_W is the combination factor for wind action, $\psi_W = 0.4$

Durability

2.6(1)

The design working life for the mast is presented in the execution specification. The design working life for fatigue should be determined according to standard SFS-EN 1993-1-9 and its National Annex.

The recommended design working life for important radio and TV masts and telephone/link masts is 50 years. The recommended design working life for other structures (mobile phone network base station masts, lighting masts etc.) is 30 years.

Allowance for corrosion

4.1(1), Note 1
See also standard SFS-EN ISO 10684 concerning the hot-dip-galvanising of bolts.

Guys

4.2(1)
When assessing the need for possible protection measures, the design working life of the structure should be taken into account. The replacement of guys can be considered as an alternative to the protection methods recommended above.

General

6.1(1), Note 1
The following values of γ_M are used:

$\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$ $\gamma_{M2}=1.25$ $\gamma_{Mg} = 1.40$ $\gamma_{Mi} = 2.00$.

The strength of the guy assembly (guy with end fittings) decreases when bending guy wire around the fittings (wedge clamp, thimble etc.). The resistance design value of the assembly should be calculated from the expression:

$$R_{d,g} = K_e \cdot R_{k,g} / \gamma_{Mg} \quad (1.3)$$

where:

$R_{d,g}$ is the design value for guy assembly resistance

$R_{k,g}$ is the design value for guy resistance

K_e is the reduction factor that depends on the characteristics of the guy fasteners

γ_{Mg} is the partial factor for the guy.

The real resistance of the guy assembly can be demonstrated either with laboratory tests or calculations based on standard SFS-EN 1993-1-11. In the absence of tests or calculations, the values presented in Table 1 may be adopted for the reduction factor K_e:

Table 1. Reduction factor K_e for the resistance of the guy assembly

End type	K_e	Note:
Cast end	1.00	
Wedge clamp	0.80	Manufacturer recommended type suitable for guy size in question
Thimble	0.80	Manufacturer recommended type suitable for guy size in question
Other	0.70	Plug etc.

Grounds

7.1(1)
The allowable values for the deformations are defined in the execution specification. The calculations are done for reduced wind loads without ice, if other additional requirements are not specified in the execution specification. If the partial load method is used in the calculations for a guyed mast, it should also be applied to the deformation analysis at the serviceability limit state.

The combination of actions is represented in the following expression:

$$\gamma_G \cdot G_k + 0.64 \cdot \gamma_W \cdot Q_{k,w} \qquad (1.4)$$

where:

G_k is the characteristic value for self-weights of the structure and equipment
$Q_{k,w}$ is the characteristic value for wind actions (including wind loads due to persons)
γ_G is the partial factor for self-weight, $\gamma_G = 1.0$
γ_W is the partial factor for wind actions, $\gamma_W = 1.0$

The value $\gamma_M = 1.0$ is adopted for the partial factor for materials.

Partial factors for fatigue

9.5(1)
Values according to the National Annex to standard SFS-EN 1993-1-9 are adopted.

Reliability differentiation for towers and masts

A.1(1)
Instead of reliability classes, the consequences classes pursuant to the National Annex to standard SFS-EN 1990 presented in Ministry of Environment Guideline: Basis of structural design for load-bearing structures, regulations and instructions are used.

The reliability classes 1, 2 and 3 presented in standard SFS-EN 1993-3-2 correspond to the consequences classes CC1, CC2 and CC3 presented in the Ministry of Environment Decree concerning standard SFS-EN 1990.

Partial factors for actions

A.2(1)P, Note 2

The partial factors for variable actions γ_Q are given in Table 2.

Table 2. Partial factors for permanent and variable actions

Effect of action	Consequences class	Permanent actions	Variable actions
Unfavourable	3	1.2	1.4
	2	1.1	1.2
	1	1.0	1.1
Favourable	All classes	1.0	0.0
Accidental situations		1.0	1.0

Ice load

C2(1)

The type of the ice used for tower and mast structures is rime ice, see ISO 12494, clause 7.5.

The mast structure should be divided vertically into sections with a maximum height of 100 m. The ice class will be defined for each section by using the height level at 2/3 of the height of the section, measured from the bottom of the section. The ice class for a guy can be assumed to be constant across the entire length of the guy. The ice class is defined at a height of 2/3 of the height of the guy attachment level.

If no more accurate information is available, the following assumptions may be used:

- The ice class and the relevant ice weight on structural elements at a certain height are defined according to Table 3 of this National Annex. The values in Table 3 are based on an ice density of 300 kg/m^3 for the elements in the mast shaft and 400 kg/m^3 for the guys.
- When calculating the thickness of the ice deposit on an element in a tower or mast for the determination of the effective wind area, the use of the principles in ISO 12494 is recommended. Clause C.6 of this National Annex presents an alternative simplified method.

- The force coefficient for iced single construction elements and guys can be obtained from Tables 17 to 25 of ISO 12494 (see also Table B.2.1 in Annex B to SFS-EN 1993-3-1).

For masts in consequences class 3 with ice class R6 or higher, the eccentric ice in the shaft and asymmetrical icing of the guys should be considered. The centre of the eccentric shaft ice is assumed to be at a distance of 0.5 times the shaft width from the shaft centre in the most unfavourable direction in each load case for the structural element concerned. In the cases of asymmetrical icing of the guys, one or more guys are without ice according to Table 5.

The force coefficient of an iced lattice shaft is based on the solidity ratio of the mast faces according National Annex B of standard SFS-EN 1993-3-1. In this case, the parameters to be used are determined on the basis of the shape of the chord of lattice shaft in a manner where all the shafts are assumed to have the same shape. Furthermore, the force coefficient calculated in this manner is multiplied by a correction factor that takes the shape of the ice into account; this is given in Table 6.

The ice load of a fully iced lattice shaft is determined with the help of the ice layer thickness provided in the execution specification, or by calculating it from the expression (1.6) in this National Annex (value $T_{i,g}$) and assuming that the icing is symmetrical.
Falling ice should be considered according to ISO 12494, Chapter 11.

Table 3. Ice loads and *k* factors in different ice classes

Ice class	H (m)	g_i (kg/m)	k
R1	0–50	0.5	0.40
R2	50–100	0.9	0.45
R3	100–150	1.6	0.50
R4	150–200	2.8	0.55
R5	200–250	5.0	0.60
R6	250–300	8.9	0.70
R7	300–350	16.0	0.80
R8	350–400	28.0	0.90
R9	400–450	50.0	1.00

H is the relative height from the average level of the surrounding terrain within a distance of 10 km from the site
g_i is the characteristic value of the ice weight on the structural element
k is the reduction factor for wind and ice combinations (see clause C.6 of standard SFS-EN 1993-3-1)

Table 4. Force coefficient for fully iced lattice shaft $C_{f,S,0,i}$

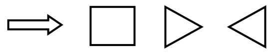
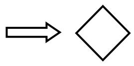
Ice class	Force coefficient for fully iced shaft $C_{f,S,0,i}$	
		
R1–R3	2.0	1.8
R4–R5	1.8	1.6
R6–R7	1.6	1.4
R8–R9	1.4	1.2
Wind drag is calculated for the area projected perpendicular to the wind		

Table 5. Asymmetrical ice loads for guys, where N is the number of the guy level

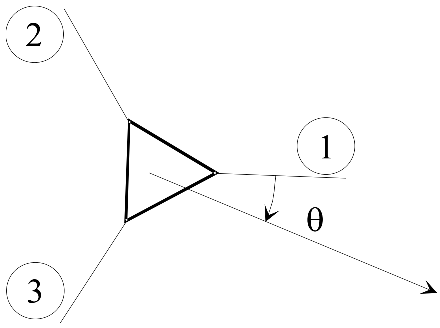
Case	Wind direction	Guys without ice	Wind and guy directions
	180	All guys in direction 1	θ = wind direction 
2	0	All guys in directions 2 and 3	
3	0	Guys in directions 2 and 3 at guy level 1	
Na	0	1_N, 2_{N-1}, 3_{N-1}	
Nb	0	1_N, 2_{N-1}, 3_{N-1}, 2_{N+1}, 3_{N+1}	
Key: 2_{N-1} refers to guy in direction 2 at guy level N-1			

Table 6. Correction factor of the resistance of the iced lattice shaft

Ice layer thickness $T_{i,s}$	Correction factor of the resistance of the iced lattice shaft. Intermediate values can be interpolated.	
B_0 = width of chord	Round chords	Angular chords
$T_{i,s} \leq B_0/8$	1.10	1.00
$T_{i,s} \geq B_0/2$	1.30	0.74

Combinations of ice and wind

C.6(1)

The values of factor *k* defined in standard ISO 12494 are given in Table 3. The following combination factors are used:

$\psi_w = 0.5$ (1.5)

$\psi_{ice} = 0.3$

The wind area of an iced structural member is calculated using the following design values for ice weight:

$g_{i,d} = g_i$ in expression C.1. g_i is taken from Table 3.

$g_{i,d} = \psi_{ice}\, g_i$ in expression C.2.

Instead of using standard ISO 12494, the ice layer thickness in the structural member or continuous ancillary that is used in the wind surface calculation may be alternatively calculated with a simplified method from expression (1.6). The ice layer is assumed to have uniform thickness across all sides of the structural member.

$$T_{i,s} = \sqrt{\frac{4G_{i,d}}{3\rho_i} + B^2} - B \qquad T_{i,g} = T_{i,s}/\,2 \qquad (1.6)$$

where:

$T_{i,s}$	is the thickness of the ice layer on an element or ancillary in the lattice structure
$T_{i,g}$	is the thickness of the ice layer on a guy
$G_{i,d}$	is the design value for the ice weight (G_i will be taken from Table 2)
ρ_i	is the density of the ice
B	is the width of the structural member or the diameter of the guy without ice.

Expression (1.6) is valid for structural members with width $B \leq 300$ mm. For larger structural members and compact tubular shafts, the ISO 12494 method for single elements should be used. The difference between the values of $T_{i,g}$ and $T_{i,s}$ is due to the symmetrical icing on the guy.

Temperatures at different load conditions:

- Reference condition (no wind, no ice) 0°C
- Wind, no ice -20°C
- Wind and ice (all combinations) 0°C.

The temperature is taken into account when determining the air density for the wind pressure.

Guys

D.1
Rope safety clamps should not be used for the attachment of the guy ropes.

Insulators

D.3(6), Note 1
The breakage of a guy insulator shall not cause the collapse of the mast.

Lightning protection

D.4.2(3)
The leg joints of the structure shall be provided with a good galvanic connection. The towers and masts should be equipped with a ground wire (minimum size 25 mm^2 copper or 50 mm^2 steel) from the top to the base of the structure. It shall be connected to an underground radial earthing net which should fulfil the appropriate requirements of the authorities and the client.

Flight obstacle warnings

D.4.3(1)
A structure that is considered a flight obstacle should be painted with flight obstacle colours and/or equipped with flight obstacle lights according to the requirements of ICAO and the national aviation authority.

Details are given in the Finnish Civil Aviation Administration's decision no. 1/2000. The details on the markings can be found in the aviation regulation AGA M3-6.

National Annex to standard SFS-EN 1993-3-2 Part 3-2: Towers, masts and chimneys. Chimneys

As regards standard SFS-EN 1993-3-2, the recommended values set forth in standard SFS-EN 1993-3-2 and all the annexes to standard SFS-EN 1993-3-2 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 1993-3-2:
– 2.3.3.1(1), Note 1
• 2.3.3.5(1), Note 1
• 2.6(1)
– 4.2(1)
– 5.1(1)
– 5.2.1(3)
– 6.1(1)P
• 6.2.1(6)
– 6.4.1(1)
– 6.4.2(1)
– 6.4.3(2), Note 1
– 7.2(1)
– 7.2(2), Note 2
– 9.1(3)
– 9.1(4)
– 9.5(1)
• A.1(1)
• A.2(1), Note 2
• A.2(1), Note 3
• C.2(1).

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

Ice loads

2.3.3.5(1), Note 1
Ice loading should be determined based on the local conditions for each project. The combination factor ψ should be determined according to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990. . For the chimney's bracing structure, when bracing structure is a truss structure, the ψ value of National Annex of standard SFS-EN 1993-3-1 can be used.

Durability

2.6(1)
The planned service life of the structure should be determined separately for each project.

External corrosion allowance

4.2(1)
For coated structures, the relevant standards should also be used. A normal environment means classes C1, C2 or C3 according to SFS-EN 12944.

Demonstration of strength

6.2.1(6)
In fatigue loaded structures, the distribution of stresses mentioned above should be taken into account case by case. See also chapter 9 of standard SFS-EN 1993-3-2.

Reliability differentiation for steel chimneys

A.1(1)
Instead of reliability classes, the consequences classes pursuant to the National Annex to standard SFS-EN 1990 presented in Ministry of Environment Guideline: Basis of structural design for load-bearing structures, regulations and instructions are used.

The reliability classes 1, 2 and 3 presented in standard SFS-EN 1993-3-2 correspond to the consequences classes CC1, CC2 and CC3 presented in the Ministry of Environment Decree 3/16 concerning the application of standard SFS-EN 1990.

Partial factors for actions

A.2(1), Note 2
The combinations of actions pursuant to Decree 3/16 on the application of standard SFS-EN 1990 and the actions pursuant to the Decree on the application of standard SFS-EN 1991 are used.

A.2(1), Note 3
The Ministry of Environment Decree 7/16 on the application of standard SFS-EN 1991-1-4 and the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are followed.

Increase in fatigue strength resulting from special requirements

C.2(1)

The use of higher fatigue classes than those given in the standard SFS-EN 1993-1-9 should be based on reliable testing according to Annex D of standard SFS-EN 1990.

National Annex to standard SFS-EN 1993-4-1 Part 4-1: Silos

As regards standard SFS-EN 1993-4-1, the recommended values set forth in standard SFS-EN 1993-4-1 and all the annexes to standard SFS-EN 1993-4-1 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 1993-4-1:

- • 2.2(1)
- – 2.2(3), Note 1
- – 2.9.2.2(3)
- • 3.4(1)
- • 4.1.4(2)
- • 4.1.4(4), Note 1
- – 4.2.2.3(6)
- – 4.3.1(6)
- – 4.3.1(8)
- – 5.3.2.3(3)
- – 5.3.2.4(10)
- – 5.3.2.4(12), Note 1
- – 5.3.2.4(15)
- – 5.3.2.5(10)
- – 5.3.2.5(14)
- – 5.3.2.6(3)
- – 5.3.2.6(6)
- • 5.3.2.8(2)
- – 5.3.3.5(1)
- – 5.3.3.5(2)
- – 5.3.4.3.2(2)
- – 5.3.4.3.3(2)
- – 5.3.4.3.3(5)
- – 5.3.4.3.4(6)
- – 5.3.4.5(3)
- – 5.4.4(2)
- – 5.4.4(3b)
- – 5.4.4(3c)
- – 5.4.7(3)
- – 5.5.2(3)
- • 5.6.2(1)
- • 5.6.2(2)
- – 6.1.2(4)

- 6.3.2.3(2)
- 6.3.2.3(4)
- 6.3.2.7(4), Note 1
- 7.3.1(4)
- 8.3.3(4)
- 8.4.1(6), Note 1
- 8.4.2(5), Note 1
- 8.5.3(3)
- 9.5.1(3)
- 9.5.1(4)
- 9.5.2(5), Note 1
- ● 9.8.2(1)
- ● 9.8.2(2)
- A.2(1)
- A.2(2)
- A.3.2.1(6)
- A.3.2.2(6)
- A.3.2.3(2)
- A.3.3(1)
- A.3.3(2)
- A.3.3(3)
- A.3.4(4).

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

Reliability differentiation

2.2(1)
The consequences classes are determined according to the National Annex to standard SFS-EN 1990 presented in Ministry of Environment Guideline: Basis of structural design for load-bearing structures, regulations and instructions are used.

2.2(3)
As regards the analysis methods (see clause 4.2.2 of standard SFS-EN 1993-4-1) and other design instructions, however, the requirements, methods and instructions presented for different consequences classes in standard SFS-EN 1993-4-1 are applied. Standard SFS-EN 1993-4-1 uses the term "consequences class" in a slightly different meaning than the Ministry of Environment Decree 3/16 concerning the application of standard SFS-EN 1990. Consequences class in standard SFS-EN 1993-4-1 also describes, in part, the difficulty of design and the precision required in the analysis.

Partial factors for actions on silos

2.9.2.1(1)P

The partial factors for actions on silos are determined according to the Ministry of Environment Decree concerning standard SFS-EN 1991-4. The load combinations are determined pursuant to the Ministry of Environment Decrees 3/16 and 12/16 concerning application of standard SFS-EN 1990 and standard SFS-EN 1991-4 SFS-EN 1991-4.

Special alloy steels

3.4(1)

The mechanical properties of non-standardised alloy steels should be reliably determined on a case-by-case basis.

Allowance for corrosion and abrasion

4.1.4(2)

Appropriate values should be reliably determined on a case-by-case basis.

4.1.4(4) Note 1

Appropriate values should be reliably determined on a case-by-case basis.

Fatigue, LS4

5.3.2.8(2)

The recommended value should be used unless another value is required due to the use of the silo. See standard SFS-EN 1993-1-9.

Deflections

5.6.2(1)

The characteristic load combinations pursuant to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are used. The recommended value is used for the limiting value of deflection.

5.6.2(2)

The characteristic load combinations pursuant to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are used. The recommended value is used for the limiting value of deflection.

Deflections

9.8.2(1)
The characteristic load combinations pursuant to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are used. The recommended value is used for the limiting value of deflection.

9.8.2(2)
The characteristic load combinations pursuant to the Ministry of Environment Decree 3/16 on the application of standard SFS-EN 1990 are used. The recommended value is used for the limiting value of deflection.

National Annex to standard SFS-EN 1993-4-2 Part 4-2: Tanks

As regards standard SFS-EN 1993-4-2, the recommended values set forth in standard SFS-EN 1993-4-2 and all the annexes to standard SFS-EN 1993-4-2 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 1993-4-2:

- • 2.2(1)
- – 2.2(3)
- • 2.9.2.1(1)P
- • 2.9.2.1(2)P
- • 2.9.2.1(3)P
- – 2.9.2.2(3)P
- – 2.9.3(2)
- – 3.3(3)
- – 4.1.3(7)
- – 4.1.4(3).

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

Reliability differentiation

2.2(1)
The consequences classes are determined according to the National Annex to standard SFS-EN 1990 presented in Ministry of Environment Guideline: Basis of structural design for load-bearing structures, regulations and instructions are used.

2.2(3)
Standard SFS-EN 1993-4-2 uses the term “consequences class” in a slightly different meaning than the Ministry of Environment Decree 3/16 concerning the application of standard SFS-EN 1990. Consequences class in standard SFS-EN 1993-4-2 also describes, in part, the difficulty of design and the precision required in the analysis.

As regards the analysis methods (see clause 4.2.2 of standard SFS-EN 1993-4-2) and other design instructions, however, the requirements, methods and instructions presented for different consequences classes in standard SFS-EN 1993-4-2 are applied.

Standard SFS-EN 1991-4 does not deal with tanks that are serially manufactured in a factory. Tanks that are serially manufactured in factories are designed in the same manner as other tanks.

Partial factors for actions on tanks

2.9.2.1(1)P
The partial factors for actions pursuant to standards SFS-EN 1990 and SFS-EN 1991-4 and the Ministry of Environment Decrees 3/16 and 12/16 concerning their application are used in persistent and transient design situations.

2.9.2.1(2)P
The partial factors for variable actions pursuant to standards SFS-EN 1990 and SFS-EN 1991-4 and the Ministry of Environment Decrees 3/16 and 12/16 concerning their application are used in accidental situations.

2.9.2.1(3)P
Similar partial factors as for other tanks are used for tanks that are serially manufactured in a factory.

Allowance for corrosion

4.1.3(7)
Corrosion allowance is determined case-by-case on the basis of conditions and presented in the design brief.

National Annex to standard SFS-EN 1993-5 Part 5: Piling

As regards standard SFS-EN 1993-5, the recommended values set forth in standard SFS-EN 1993-5 and all the annexes to standard SFS-EN 1993-5 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 1993-5:
– 3.7(1)
• 3.9(1)P
• 4.4(1)
– 5.1.1(4)
• 5.2.2(2), Note 2
– 5.2.2(13)
– 5.2.5(7)
– 5.5.4(2)
• 6.4(3), Note 1
– 7.1(4)
– 7.2.3(2), Note 1
• 7.4.2(4)
• A.3.1(3)
• B.5.4(1), Note 1
– D.2.2(5).

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

Fracture toughness

3.9(1)P
The lowest service temperature is determined according to the Ministry of Environment Decree concerning standard SFS-EN 1991-1-5. The fracture toughness should be checked at all operating temperatures with a relevant load case corresponding to that temperature. Safety against brittle fracture should be checked for the installation stage and the final structure.

Corrosion rates for design

4.4(1)
The instructions given in Table 4.1 or 4.2 should be used if local conditions do not require other values. The applicability of the values given in Tables 4.1 and 4.2 should be stated based on preliminary investigations and on previous knowledge and experience of the

soil in cases where there are no reasons to assume that the soil or water is polluted. In unclear cases, the research programme should be refined.
Tables 4.1 and 4.2 do not apply to stainless steels.

Sheet piling in bending and shear

5.2.2(2), Note 2
The numerical value for β_B should be reliably determined for each project.

Structural aspects of steel sheet piling

6.4(3), Note 1
The numerical value for β_D should be determined reliably for each project depending on, for example, the profile and the used locking.

Ultimate limit state verification

7.2.3(2)
In addition, the fabrication method of the threads should be taken into account according to clause 3.6.1(3) of standard EN 1993-1-8.

Load-bearing piles

7.4.2(4)
For impact driven piles and drilled piles, the following provisions should be followed:

a) The characteristic compression, tension and bending resistance of the splice of the pile should fulfil the requirements of Table 1 when the splice is tightened.

b) The bending resistance and bending stiffness of the splice should be tested according to the test arrangements given in Figure 1. For the splice of an impact driven pile, the tests should be made after the impact test. For the splice of a drilled pile, the test may be made after the tightening of the splice.

Table 1. Requirements for resistances and bending stiffness of impact driven and drilled pile splices

Characteristic value of compression resistance	Characteristic value of tension resistance	Characteristic value of bending resistance	Bending stiffness *EI* (0.3…0.8·$M_{k,pile}$)
> $N_{k,pile}$	> 0.15·$N_{k,pile}$	> $M_{k,pile}$	> 0.75·$EI_{p,pile}$

where:

$N_{k,pile}$ is the characteristic compression resistance of the steel part of the pile, when corrosion allowance is not taken into account;

$M_{k,pile}$ is the characteristic bending resistance of the steel part of the pile, when corrosion allowance is not taken into account;

$EI_{p,pile}$ is the characteristic bending stiffness of the steel part of the pile, when corrosion allowance is not taken into account.

Bending stiffness *EI* [kNm^2] may be calculated from:

$$EI = \frac{M \cdot l^2}{8 \cdot \delta} \quad (1.1)$$

where:

M is the bending moment [kNm];

l is the distance between measurement points [m];

δ is the deflection of the pile between measurement points [m].

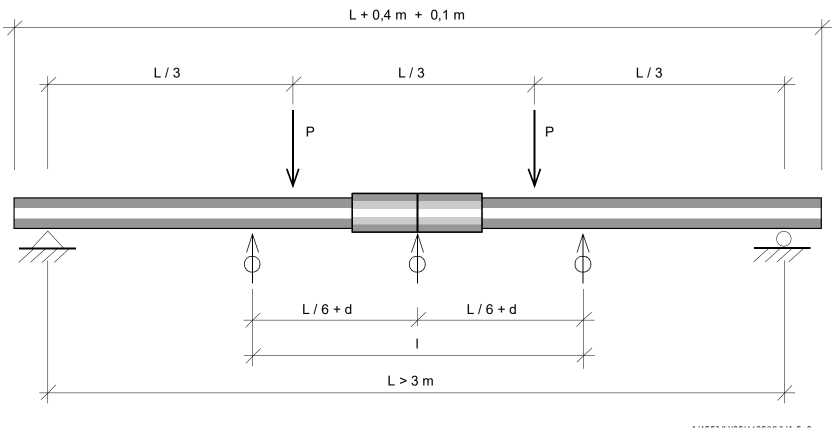


Figure 1. *Bending test for spliced pile*

Material properties

A.3.1(3)
Values according to the National Annex to standard SFS-EN 1993-1-1 are followed for material properties.

Design values

B.5.4(1), Note 1
The value of factor η_{sy} should be determined reliably in each project.

National Annex to standard SFS-EN 1993-6 Part 6: Crane supporting structures

As regards standard SFS-EN 1993-6, the recommended values set forth in standard SFS-EN 1993-6 and all the annexes to standard SFS-EN 1993-6 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 1993-6:

- • 2.1.3.2(1)P
- – 2.8(2)P
- • 3.2.3(1)
- – 3.2.3(2)P
- – 3.2.4(1), Note 2
- • 3.6.2(1)
- • 3.6.3(1)
- – 6.1(1)
- • 6.3.2.3(1)
- • 7.3(1)
- – 7.5(1)
- – 8.2(4)
- • 9.1(2)
- – 9.2(1)P
- • 9.2(2)P
- – 9.3.3(1)
- – 9.4.2(5).

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

Planned service life

2.1.3.2(1)P
The planned service life should be determined project by project.

Fracture toughness

3.2.3(1)
The lowest service temperature should be determined project by project, taking into account the planned service life of the structure.

Rail steels

3.6.2(1)
The information for rails and rail steels should be given project by project in the execution specification.

Special connecting devices for rails

3.6.3(1)
The information for special connecting devices for rails should be given project by project in the execution specification.

Assessment methods

6.3.2.3(1)
The method presented in Annex A is used as an alternative method.
When using method A to determine χ_{LT}, the National Annex to standard SFS-EN 1993-1-1 is taken into account.

Limits for deformations and displacements

7.3(1)
At the serviceability limit state, the limit value used for deformations and displacements is the recommended value pursuant to the clause in the standard, unless the use of a crane or other reasons, such as the crane operator's travelling along with the crane, require the use of lower values. If necessary, a limit for the rotation of the runway beam shall be presented.

Requirements for fatigue assessment

9.1(2)
The value for cycle count C_0 is 0.

Partial factors for fatigue

9.2(2)P
The instructions provided in the National Annex to standard SFS-EN 1993-1-9 shall be applied.

Annex A

Alternative assessment method for determining lateral torsional-buckling resistance

χ_{LT} *The determination shall take into account the National Annex to standard SFS-EN 1993-1-1.*