**BFS 1993:58 with amendments up to BFS 1998:39,
BFS 1999:7 and BFS 1999:46**

DESIGN REGULATIONS BKR

Mandatory provisions and general recommendations

BOVERKET
Swedish Board of Housing, Building and Planning

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Summary: This reprint of Design Regulations, BKR, of the Swedish Board of Housing, Building and Planning contains mandatory provisions and general recommendations pursuant to the Planning and Building Act (1987:10), PBL, the Act on Technical Requirements for Construction Works etc (1994:847) and the Decree on Technical Requirements for Construction Works etc (1994:1215).

This edition of BKR has inter alia been adapted to new and amended standards. Other important changes are the new classification of frost resistance of bricks and the introduction of Class D Mortar. Necessary clarifications and some other adjustments have also been made.

Key words: Building regulations, design regulations, regulations for new buildings, mandatory provisions, general recommendations, structures, resistance, durability, strength, actions, design, geotechnics, stability, timber structures, masonry, concrete structures, steel structures, aluminium structures, fire resistance, properties, supervision and control, calculation methods, formulae, planning and building act, act on technical requirements for construction works etc, the properties of buildings, BBR 94, BKR 94, BBR, BKR, NR, PBL, BVL, BFS.

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FÖRFATTNINGSSAMLING

BFS 1998:39
BKR 3

Utgivare: Peggy Lerman

Provisions of the Swedish Board of Housing, Building and Planning regarding amendments to the Board's Design Regulations (mandatory provisions and general recommendations);

Published
13 November 1999

resolved by the Executive Board of Boverket on 11 August 1998 after consents of the Government according to 27 § 4 Government Agencies and Institutes Ordinance (1995:1322).

Information procedures according to Ordinance (1994:2029) on information procedures for technical regulations have been accomplished[1].

The National Board of Housing, Building and Planning prescribes the following regarding the Board's Design Regulations (BFS 1993:58)[2] pursuant to section 19 of the Planning and Building Decree (1987:383) and Section 18 of the Decree (1994:1215) on Technical Requirements for Construction Works etc,

that Section 3:62 is deleted,

that the headings 3:61 and 3:62 are deleted,

that the designation "BKR 94" is replaced by "BKR",

that the section closest to the heading in Section 10, Sections 1:1, 1:4, 1:5, 2:114, 2:115, 2:12, 2:13, 2:21, 2:32, 2:321, 2:322, 2:5, 2:61, 3:2, 3:3, 3:41, 3:431, 3:432, 3:5, 3:6, 4:22, 4:3121, 4:5, 4:6, 5:23, 5:241, 5:243, 5:244, 5:311, 5:3121-5:3123, 5:322, 5:41, 5:411-5:413, 5:42, 6:11, 6:21, 6:211, 6:213, 6:214, 6:22, 6:311, 6:3121, 6:3122, 6:3124, 6:3128, 6:3129, 6:4, 6:41-6:45, 6:51, 6:611, 7:222, 7:223, 7:231, 7:24, 7:3125, 7:32, 7:43, 8:221, 8:225, 8:3124, 8:43, 9:221, 10:1 and 10:221 and the heading of Section 2:5 shall have the following wording,

that a new section is added, with a new heading, 5:415 with the following wording,

that a new text is added closest to the heading 6:62 with the following wording.

The Statute will thus read as follows as from the day this Statute came into force.

Transitional regulations[3]

Building Regulations (BFS 1993:57), BBR 94, and Design Regulations (BFS 1993:58), BKR 94, of the Board come into force on 1 January 1994 when the Board's Regulations for New Buildings (BFS 1988:18 as amended by 1990:28, 1991:38 and 1993:21) cease to apply.

1 See the Directive of the Council 83/189/EEC of 28 March 1983, EGT number L 109, 26.4.1983, page 8 (Celex 383L0189).

2 The Statute is amended and reprinted 1995:18.

3 To BFS 1993:58. Latest wording BFS 1995:18.

Previous regulations shall however apply for work subject to a permit requirement for which an application for a permit is submitted prior to 1 January 1994 and a decision regarding this is made by the municipality before 1 July 1995, and for work which does not require a permit and which is commenced before 1 January 1994.

If the applicant so requests, previous regulations shall be applied for work for which a permit application is submitted before 1 January 1995 and a decision regarding this is made by the municipality before 1 July 1995. The new mandatory provisions in Sections 1:4 and 1:5 of BBR 94 and BKR 94 shall however be applied.

This Statute[4] comes into force on 1 July 1995. Previous regulations shall however be applied in conjunction with applications in regard to which a decision has been made by the municipality prior to 1 July 1995.

This Statute[5] comes into force on 1 January 1999. Previous regulations shall however apply for work subject to a building notification for which a building notification is submitted prior to 1 January 1999, and for work which does not require a building notification and which is commenced before 1 January 1999.

FREDRIK VON PLATEN

Sture Åkerlund
(Building Division)

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[4] BFS 1995:18.
[5] BFS 1998:39.

1 INTRODUCTION

1:1[6] General

This Statute contains mandatory provisions and general recommendations pursuant to the Planning and Building Act (1987:10), PBL, the Act (1994:847) on Technical Requirements for Construction Works etc, BVL, and the Decree (1994:1215) on Technical Requirements for Construction Works etc, BVF (*the principal Statutes*).

General recommendation:

Further mandatory provisions and general recommendations regarding the characteristics of buildings are given in the Building Regulations of the Board (BFS 1993:57), BBR. *(BFS 1998:39)*

Further regulations regarding type approval etc are given in the mandatory provisions and general recommendations of the Board regarding type approval and production control (BFS 1995:6).

1:2 Mandatory provisions

The mandatory provisions apply
– when a building is constructed,
– with regard to parts of the building which are constructed when the building is added to,
– earthworks and demolition works, and
– sites which are used for building development.

General recommendation:

It is evident from Section 14 Paragraph 2 of BVF that, in applying the requirements to additions, consideration shall be given to the extent of the additions and the condition of the building.

It follows from Section 18 of BVF that other authorities may also have the right to issue mandatory provisions regarding the design etc of buildings. Examples of these are the provisions of the Swedish Board of Occupational Safety and Health regarding special work environmental aspects and the provisions of the Swedish Board of Agriculture regarding the design of livestock buildings. *(BFS 1995:18)*

If there are special reasons for this and the construction project may nevertheless be assumed to be technically satisfactory and there is no appreciable inconvenience from some other standpoint, the Building Committee may in individual instances permit minor deviations from the mandatory provisions in this Statute.
(BFS 1995:18)

[6] Latest wording BFS 1995:18

General recommendation:

The Building Committee may, in accordance with Chapter 9 Section 8 of PBL, publish details of its views in the minutes of the building committee meeting. *(BFS 1995:18)*

1:3 General recommendations

The general recommendations regarding the application of the mandatory provisions in this Statute and in the principal Statutes indicate how someone *can* or *should* act in order to comply with the requirements of the mandatory provisions. The individual is however at liberty to select other technical solutions and methods if these comply with the requirements of the mandatory provisions.

The general recommendations may also contain certain explanatory or editorial information.

The general recommendations are preceded by the word *General recommendations* and are indented and printed in a smaller type immediately after the mandatory provision to which they refer.

1:4[7] Type approval and production control

The term *type approved* or *production controlled* materials and products refers to materials, structures or arrangements which have been type approved or subjected to production control in accordance with Sections 18 - 20 of BVL. Construction products which have been shown to satisfy the requirements in Sections 4 and 5 of BVL are considered to be of equal status.

The term *production controlled* materials and products in this Statute also refers to materials and products which have been subjected to control by the following bodies in relation to the control regulations referred to:

– Concrete and Aggregate Certification (BBC) AB, in relation to BBK 94 Sections 9.3 - 9.5 and the BBC application regulations 1995-1996,

– Swedish Institute for Supervision of Steel Products in Building, SBS, in relation to BBK 94, Section 9.4, the *SBS Application Regulations*, 1986, *Special Regulations for Manufacturers of Steel Structures, D*, 1986 and *Special Regulations for Wholesalers/Importers, G*, 1986. In the *BBC Application Regulations* "SBS" may be replaced by "a certification body accredited for the relevant task". *(BFS 1998:39)*

1:5[8] Standards

Methods and design solutions set out in European Standards adopted as Swedish Standards (SS-EN), and in European Prestandards (SS-ENV), are approved as alternatives to the methods

[7] Latest wording BFS 1995:18. The amendment means inter alia that the second, fourth and fifth items are deleted.

[8] Latest wording BFS 1995:18.

and design solutions set out in this Statute, subject to the limitations and other conditions which may be specified in the Board's regulations relating to the standard concerned. Such regulations are published in the Board's regulation series BFS/NAD. These regulations shall be applied even when reference is made in this Statute to a European Standard or European Pre-standard which has been adopted as a Swedish Standard.

When designing an individual structural element or interacting elements, either BKR or SS-EN (or SS-ENV) including any associated NAD shall be applied. *(BFS 1998:39)*

General recommendation:

The consequence of the above is, for example, that

– the design of a slab element, i.e. bending, shear etc.
– control of the overall stability

either is undertaken according to BKR or according to SS-EN or SS-ENV including any associated NAD. *(BFS 1998:39)*

When designing according to SS-EN or SS-ENV including any associated NAD, also the structural detailing shall be undertaken according to the relevant SS-EN or SS-ENV and its associated NAD. *(BFS 1998:39)*

1:6 Terminology

Terms which are not specifically defined in the principal Statutes or in the mandatory provisions in this Statute have the meaning set out in Publication No TNC 95, *Glossary of Planning and Building Terms 1994.*

1:7 Further information

The standards, regulations, other documents etc to which these mandatory provisions and general advisory notes refer are listed in a schedule appended to this Statute, *Appendix*. As and when applicable, the edition of the standard etc to which reference is made is also given in the schedule.

[10] Latest wording BFS 1995:18.

2 GENERAL REGULATIONS FOR LOADBEARING STRUCTURES

2:1 Requirements

2:11 Requirements in the ultimate limit states

:111 Material failure and instability

Loadbearing structures shall be designed and detailed so that an adequate degree of safety is provided with respect to material failure and instability in the form of lateral and local buckling, lateral instability and similar during the construction of the structure, its service life and in the event of fire.

General recommendation:

Failure or instability may also occur due to deformations in the supporting ground.

:112 Tilting, uplift and sliding

Buildings and their parts shall be designed and detailed so that an adequate degree of safety is provided with respect to tilting, uplift and sliding.

:113 Accidental actions and progressive collapse

Buildings shall be designed so that the risk of progressive collapse is slight. This may be accomplished by designing and detailing buildings either in such a way that they can withstand accidental actions or in such a way that primary damage is limited. Such damage shall not give rise to progressive collapse and severe destruction in any part of the structure other than the region of primary damage and the region adjoining this. *(BFS 1995:18)*

Special measures need not be taken in buildings in which the risk of serious accidents due to progressive collapse is slight, or in buildings which are so small that primary damage causes total destruction.

General recommendation:

The requirement relating to accidental actions and progressive collapse normally applies only to elements of structure assigned to Safety Class 3. See Boverket’s handbook *Vibration, induced deformation and accidental actions.*

A stairway which constitutes the only escape route in a building shall at all times be designed for accidental actions.

:114 Safety index

The safety index β, as defined in accordance with ISO 2394-1998, *General Principles on the Reliability for Structures*, shall have the following values for a structural element:
$\geq$ 3.7 for Safety Class 1
$\geq$ 4.3 for Safety Class 2
$\geq$ 4.8 for Safety Class 3 *(BFS 1998:39)*

In design with respect to accidental actions and the risk of progressive collapse, the value of the safety index β shall be not less than 3.1 and 2.3 respectively.

General recommendation:

The above values of β relate to a reference period of 1 year.

The values of the partial factors in the ultimate limit states have been calculated with respect to the above values of β and are based on calibrations in accordance with NKB Report No. 55E *Guidelines for Loading and Safety Regulations for Structural Design*, 1987. *(BFS 1998:39)*

If a probabilistic method is used, design shall be based on the rules relating to the method of partial factors.

:115[10] Safety classes

With regard to the extent of injury to persons which the failure of an element of structure may cause, this shall be assigned to one of the following safety classes:
- Safety Class 1 (low), little risk of serious injury to persons
- Safety Class 2 (normal), some risk of serious injury to persons
- Safety Class 3 (high), great risk of serious injury to persons.

General recommendation:

In addition to the safety class requirement which relates only to injury to persons, the building owner may stipulate more stringent requirements, for instance with respect to property damage.

In selecting the safety class, the following principles shall be applied.

Elements of structure may be assigned to *Safety Class 1* if at least one of the following requirements is complied with:
- persons are present only in exceptional cases in or in the vicinity of the building,
- the element of structure is of such nature that a failure cannot reasonably be expected to cause injury to persons, or
- the element of structure has properties such that a failure does not cause collapse but only loss of serviceability.

Elements of structure shall be assigned to *Safety Class 3* if the following conditions simultaneously apply:
– the design and use of the building are such that many persons are often present in or in the vicinity of the building,
– the element of structure is of such nature that collapse involves a high risk of injury to persons, and
– the element of structure has properties such that failure causes immediate collapse.

The classification of other elements of structure shall be not lower than *Safety Class 2*.

In design in the ultimate limit states on the basis of the method of partial factors, the safety class for an element of structure shall be taken into consideration by means of the partial factor γ_n in the way set out below.
– Safety Class 1, partial factor $\gamma_n = 1.0$
– Safety Class 2, partial factor $\gamma_n = 1.1$
– Safety Class 3, partial factor $\gamma_n = 1.2$

In design with respect to
– fire
– accidental actions and the risk of progressive collapse,
the value of γ_n may be put equal to 1.0 irrespective of safety class.

One condition which must be complied with in order that the above values of the partial factor γ_n in safety classes 2 and 3, as set out in Subsection 2:115, shall be applicable is that a design check is carried out.

General recommendation:

Examples of the choice of safety class.

A *Buildings of two and more storeys, of the type residential building (with the exception of single-dwelling houses), office buildings, department stores, hospitals and schools.*

The following elements of structure should be assigned to Safety Class 3:
– The main structural system of the building inclusive of those elements of structure which are of essential importance for the stability of the system.
– Other structural elements such as columns, beams, shear panels, whose failure causes the collapse of a floor area > 150 m^2.
– Stairs, balconies, access balconies and other elements of structure which form part of the escape routes of the building.

The following elements of structure should be assigned to Safety Class 2:
– Floor beams not assigned to Safety Class 3.
– Floor slabs.
– Roof construction with the exception of lightweight stressed skin elements of non-brittle materials.
– Those parts of heavy external wall constructions (mass $\geq$ 50 kg/m^2) which are situated higher than 3.5 m above ground level and which do not form part of the main structural system of the building.
– The fixings of external wall constructions which are situated higher than 3.5 m above ground level and which do not form part of the main structural system of the building.
– Heavy partitions (mass $\geq$ 250 kg/m^2) which do not form part of the main structural system of the building.

– The fixings of heavy ceilings (mass $\geq$ 20 kg/m^2).
– Stairs which are not assigned to Safety Class 3.

The following elements of structure should be assigned to Safety Class 1:
– Lightweight (mass $\leq$ 50 kg/m^2) stressed skin elements in roofs, of non-brittle materials.
– Lightweight secondary external wall constructions of non-brittle materials.
– All secondary external wall constructions (e.g. wall studs) on the entrance storey of the building.
– Lightweight non-loadbearing partitions.
– The fixings of lightweight ceilings.
– Ground beams which do not support a wall of Safety Class 2 or 3.
– Floor slabs on or immediately above the ground.

B *Single storey buildings of the open plan type whose roofs are of large span ($\geq$ 15 m) and which are used as sports halls, exhibition halls,* places of assembly, *department stores, schools and industrial premises in which many people are present. (BFS 1998:39)*

The following elements of structure should be assigned to Safety Class 3:
– The main structural system of the building inclusive of wind bracing and the stabilising system.
– The barriers of stands etc erected where there are large differences in level and where a large number of people may be present.
– Structures which carry large overhead cranes ($\geq$ 15 m span and $\geq$ 20 tonnes lifting capacity).

The following elements of structure should be assigned to Safety Class 2:
– Roof purlins and roofing sheets which do not have a bracing or stabilising function. Purlins and sheets may be assigned to Safety Class 1 if they are fixed in such a way that the roof remains in place in the event of failure.
– The fixings of heavy roofing elements (mass $\geq$ 50 kg/m^2).
– Heavy partitions (mass $\geq$ 250 kg/m^2).
– Heavy ceilings (mass $\geq$ 20 kg/m^2).
– Girders for small hoists and overhead cranes.

The following elements of structure should be assigned to Safety Class 1:
– Secondary external wall constructions (e.g. wall studs) of not greater than 6 m height.
– Lightweight roofing elements.
– Lightweight partitions.
– The fixings of lightweight ceilings.
– Ground beams which do not support a wall of Safety Class 2 or 3.
– Floor slabs on or immediately above the ground.

C *Single-dwelling houses and other small buildings of one or two storeys.*

The main structural system and stairs of the building should be assigned to Safety Class 2. For other elements the safety classes set out in A above can be applied.

D *Single storey buildings whose roof constructions are of small span (< 15 m) and which have the same use as the buildings in B above.*

The main structural system of the building should be assigned to Safety Class 2. For other elements the safety classes set out in B above can be applied.

E *Buildings in or near which people are seldom present.*

The main structural system of the building should be assigned to Safety Class 2 and its secondary constructions to Safety Class 1, provided that the condition that people are seldom present in or near the building may with reasonable certainty be expected to continue. All loadbearing elements of structure for small buildings which are not greater than single-dwelling houses may be assigned to Safety Class 1.

F *Geostructures*

The safety class for a geo-structure depends, inter alia, on the construction above it. In certain cases, a foundation construction may be assigned to a safety class lower than that of the construction situated above.

2:12 Requirements in the serviceability limit states

General recommendation:

In addition to requirements specified in the serviceability limit states, which primarily relate only to safety and health, the building owner may stipulate more stringent requirements, for instance with respect to appearance and comfort.

If there are no other requirements, then, in design using a probabilistic method substantially in accordance with ISO 2394-1998, *General Principles on the Reliability for Structures*, the risk that the serviceability limit states will be exceeded may be put at $\beta = 1.3 - 2.3$ depending on the type of serviceability limit state. *(BFS 1998:39)*

Calculation of deformations and oscillations may be performed in accordance with the elastic theory using an analytical model which gives a reasonable description of the stiffness, mass, damping and boundary conditions of the construction.

:121 Deformation and displacement

Elements of structure and their supports shall have such stiffness that deformations or displacements of the element of structure, when used as intended, do not adversely affect its function or damage other elements of structure. Apart from the immediate deformation when an action is applied, consideration shall also be given to the effect of

– the duration of, and variations in, the action,

– the environment of the element of structure, including temperature and humidity, and

– the long term properties of the material.

:122 Oscillations

Elements of structure shall be designed so that oscillations which occur do not cause inconvenience.

:123 Cracking

Cracking in elements of structure shall be limited in view of their function and durability.

2:13 Durability

Elements of structure and materials in loadbearing structures shall either be durable, or it shall be possible for them to be protected and maintained, so that the requirements in the ultimate and serviceability limit states are complied with during the service life of the building.

If permanent protection is not possible, consideration shall be given during design to the expected changes in properties, or the structure shall be designed so that the affected parts are accessible for recurrent protective treatment.

General recommendation:

The term service life refers to the time assumed in design during which a structure which receives normal maintenance exhibits the required degree of serviceability. Unless some other values can be shown to be more correct, the design service life of structures in Safety Classes 2 and 3 should be not less than

– 50 years for structural elements which are accessible for inspection and maintenance, and
– 100 years for structural elements which are not accessible for inspection and maintenance.

The design service life of structures is the assumed average service life. *(BFS 1998:39)*

2:2 Design assumptions

2:21[11] Actions and combinations of actions

Those combinations of the effects of actions and resistance which yield the most unfavourable influence and which may occur simultaneously during construction or during the service life of the structure shall be taken into consideration.

With regard to their variation in time, actions shall be regarded as permanent or variable actions or as accidental actions.

Depending on how rapidly they are applied and the way in which the structure is affected by acceleration, actions shall be regarded as static or dynamic actions.

Actions which have so many variations that fatigue failure may occur shall be regarded as fatigue actions.

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[11] Latest wording BFS 1995:18.

General recommendation:

Normally, only the following actions need be regarded as fatigue actions:

– Dynamic actions imposed by moving parts of machinery,

– Wind action if the effect of gusts or vortex shedding is significant.

The actions imposed by cranes, overhead cranes and other materials handling equipment may be fatigue actions.

Actions which may cause significant time dependent deformations shall be regarded as actions of long duration.

General recommendation:

The following should be regarded as actions of long duration:

– All permanent actions,

– The mean value in time $\psi_1 Q_k$ of variable actions for the most unfavourable year or other appropriate period.

With regard to their distribution in space, actions shall be regarded as fixed actions or as free actions.

The values of actions shall as far as possible be determined by means of statistical methods and on the basis of results obtained empirically.

Actions which may occur simultaneously shall be combined. If, however, the probability that they occur with high values simultaneously is low, then they need not be combined.
(BFS 1998:39)

General recommendation:

The following actions need not normally be combined:

- concentrated loads and distributed imposed loads
- concentrated loads and snow loads with frequent values on structures with a span < 2 meters
- imposed loads and wind actions on barriers
- imposed loads and snow loads on balconies, stands with standing places, parking lots and basement roof slabs beneath courtyards. *(BFS 1998:39)*

Actions which have a common cause and which are highly dependent on one another and often occur simultaneously with high values shall be regarded as one single action with the same partial factor.

The characteristic value G_k of a permanent action shall be the value which has a probability of 50% of not being exceeded.

The characteristic value Q_k of a variable action shall be the value which has a probability of 98% of not being exceeded at any time during one year.

The frequent value ψQ_k of a variable action shall be determined in view of the variation of the action in time and the coefficient of variation of the action.

The characteristic value Q_{ak} of an accidental action shall be determined in view of the nature of the action.

[13] Latest wording BFS 1995:18.

For any actions which are not specified in Section 3, the value of action shall be determined in each individual case and in accordance with the principles set out in this section.

For prefabricated loadbearing elements of structure, the effects of actions which may occur during storage, transport, lifting and erection shall be taken into consideration in design.

2:22 Materials

In determining the design value of a material property, the uncertainty in the relationship between the value of the material property as determined by tests on the material and the corresponding value in the finished structure shall be taken into consideration.

The characteristic values for the strength properties of a material and for those deformation properties which affect the resistance shall be put at the lower 5% fractile unless some other value is specified in the section relating to the material concerned. For deformation properties which have no effect on resistance, the 50% fractile shall be taken.

2:23 Deviations in size and shape

Deviations in size and shape shall be taken into consideration in design if they are of significance in verifying that the requirements in the ultimate and serviceability limit states have been complied with. The dimensional deviations of individual elements of structure and the structure as a whole may be considered separately.

General recommendation:

> If several elements in compression interact, the inclination of the structure may be calculated in accordance with *Concrete Manual - Design*, Subsection 3.4:222, or in accordance with *K18*, Subsection 18:56.
> *(BFS 1995:18)*

2:3 Design by calculation and testing

(BFS 1995:18)

Design shall be carried out by calculation, testing or some combination of these. However, calculation and testing are not required if this is obviously unnecessary. *(BFS 1995:18)*

A finished structure has sufficient stiffness and robustness when swaying (oscillations), excessive cracking, deformations and similar occur only to an insignificant extent. *(BFS 1995:18)*

2:31 Calculation

Calculations shall be based on an analytical model which gives a reasonable description of the behaviour of the structure in the limit states concerned. The selected analytical model and the input parameters shall be documented.

If a certain calculation method has a high degree of uncertainty, this shall be taken into consideration. Imposed forces shall be calculated in view of the behaviour of the structure in the limit state concerned.

General recommendation:

Examples of factors which should be taken into consideration are

- the resilience of supports, end restraints, stiffeners and bracings,
- incremental forces and moments due to deformations
- action eccentricities,
- interaction between structures/elements of structures, and
- temporal effects.

2:32 The method of partial factors

In design in accordance with the method of partial factors, safety shall be taken into consideration by means of special partial factors for actions and resistance.

In using the method of partial factors, adequate safety is provided that the ultimate or serviceability limit states will not be exceeded if the following condition is satisfied.

$$S_d \leq R_d \qquad \text{(a)}$$

where $S_d = S\,(F_d, f_d, a_d, \gamma_S)$

and $R_d = R\,(f_d, a_d, C, \gamma_R)$

NOTATION

d subscript which indicates design value
S effect of action
F action
a geometrical parameter
g_S partial factor for the analytical model for the effect of action
R resistance
f material property according to Subsection 2:322
C limiting value, e.g. the greatest deformation for which the performance requirement is satisfied
γ_R partial factor for the analytical model for resistance.

General recommendation:

An alternative formulation and a more general interpretation of condition (a) is given in ISO 2394-1998, *General Principles on the Reliability for Structures. (BFS 1998:39)*

The design value of an action is

$$F_d = \gamma_f F_k \qquad \text{or} \qquad F_d = \gamma_f \psi F_k \qquad \text{(b)}$$

where F_k is the characteristic value of an action, ψ is a load reduction factor which multiplied with F_k gives the frequent value of an action, and γ_f is a partial factor according to Subsection 2:321. The product ψF_k appear in the combinations of actions. *(BFS 1998:39)*

The design resistance or limiting value in the serviceability limit states, R_d, shall be determined on the basis of the design value for the material concerned in accordance with Subsection 2:322 and the provisions of Sections 4 – 9.

:321 Design combination of actions

The values of actions set out in Section 3 shall be applied in design unless some other values can be shown to be more correct.

The combinations of actions and partial factors g_f set out in the following tables shall be applied. The value of variable action shall be put equal to zero if this produces a more unfavourable action effect.

In design with respect to fatigue, the value of g_f for variable action may be put equal to 1.0.

Table b. Prescribed combinations of actions 1–4 associated with the partial factor g_f and with the values of actions for the ultimate limit state in general

Action	Combination of actions			
	1	2	3	4
Permanent action				
Weight of elements of structure – fixed action, G_k	1.0 G_k	0.85 G_k	1.15 G_k	1.0 G_k
– free action, ΔGk	–	–	–	- 0.1 G_k
Weight of soil[1] and water at mean water level, G_k	1.0 G_k	1.0 G_k	1.0G_k	1.0 G_k
Variable action				
One variable action Q_k	1.3 Q_k	1.3 Q_k	–	–
Other variable actions, frequent value ψQ_k	1.0 ψQ_k	1.0 ψQ_k	–	–

[1] With regard to earth pressure, see Section 3:2.

General recommendation:

Combination of actions 1 is usually the design criterion.

Combination of actions 2 can be the design criterion if the weight of an element of structure is favourable and is significant for the safety of the structure, e.g. in conjunction with the uplift and tilting of structures.

Combination of actions 3 can be the design criterion if the variable actions are small in relation to the permanent actions.

Combination of actions 4 can be the design criterion if the distribution of the weight over the structure is of essential significance in relation to the effect of other actions, e.g. for moments in arches and the moment around points of zero moment in pre-stressed concrete beams.

Table b. Prescribed combinations of actions 5–7 associated with the partial factor γ_f and with the values of actions for the ultimate limit state in conjunction with accidental action, progressive collapse or fire

Action	Combination of actions		
	5	6	7
Permanent action			
Weight of elements of structure, soil and water below mean water level, G_k	1.0 G_k	1.0 G_k	1.0 G_k
Variable action			
All variable loads ψQ_k			
for which $\psi \geq$ 0.5	1.0 ψQ_k	–	1.0 ψQ_k
for which $\psi \geq$ 0.25	–	1.0 ψQ_k	–
Accidental action			
One accidental action Q_{ak}	1.0 Q_{ak}	–	–
Action in consequence of fire Q_{ak}	–	–	1.0 Q_{ak}

(BFS 1998:39)

General recommendation:

Combination of actions 5 should normally be applied only for elements of structure in Safety Class 3.

Combination of actions 6 shall be applied, after local damage, for the other part of the structure.

General recommendation:

The local damage can also occur as a consequence of fire. *(BFS 1998:39)*

Combination of actions 7 which applies in case of fire comprises a thermal action Q_{ak} which is determined either in accordance with the standard fire curve in Swedish Standard SIS 02 48 20 or on the basis of the energy balance method and the fire load density concerned. See also Section 10 and Section 5:8 of Building Regulations BBR. *(BFS 1998:39)*

Table c. Prescribed combinations of actions 8 and 9 associated with the partial factor γ_f and with the values of actions for a structure in the serviceability limit states.

Action	Combination of actions	
	8	9
Permanent actions G_k	1.0 G_k	1.0 G_k
Variable action		
One variable action with the characteristic value Q_k	1.0 Q_k	–
Other variable actions with the frequent value ψQ_k	1.0 ψQ_k	–
All variable actions with the frequent value ψQ_k	–	1.0 ψQ_k

Combination of actions 8 shall be applied in design with respect to permanent damage in the serviceability limit states.

General recommendation:

Examples of permanent damage are

– permanent deflection of beam or floor construction which may cause damage to other elements of structure or jeopardise drainage,

– permanent cracks which may affect durability.

If the cause of damage is due to actions of long duration, the term 1.0 Q_k is to be left out and ψ is to be replaced by ψ_1 according to Subsection 2:21. *(BFS 1998:39)*

Combination of actions 9 shall be applied in design with respect to temporary inconvenience in the serviceability limit states.

:322 Design values of material properties

The design values of material properties shall normally be determined from

$$f_d = \frac{k\ f_k}{h\ g_m\ g_n} \qquad \text{(a)}$$

κ	factor which is used for a material whose resistance is dependent on moisture conditions, volume under stress and the duration of the action.
f_k	characteristic value of a material property, e.g. the strength or modulus of elasticity of the material.
η	a factor which takes account of systematic differences between the material properties of a test specimen and those of a structure. Unless some other value is specified in the section dealing with the specific material, the value of η may be put equal to 1.0.
g_m	a partial factor which takes account of the uncertainty in determining resistance. Unless stated otherwise in the section dealing with the specific material, the uncertainty associated with the analytical model is normally allowed for in the value of g_m.
g_n	a partial factor which takes account of the safety class in the ultimate limit state. In the serviceability limit states, the value of g_n may be put equal to 1.0.

For some structures subjected to earth pressure, the influence of f_d in formula (a) in Subsection 2:32 is being considered in both S_d and R_d. In such cases the relevant value of g_n shall be placed on the side of the equation where the effect is the greatest and put to 1.0 on the other side. *(BFS 1998:39)*

Unless some other value is specified in the section dealing with the specific material, the value of g_m may be put equal to 1.0 in design

– for accidental action,
– with respect to progressive collapse,
– with respect to fire, and
– in the serviceability limit states.

The design resistance R_d in the event of fire shall in the ultimate limit state be determined in accordance with the provisions of Section 10.

2:33 Design by testing *(BFS 1995:18)*

The planning, execution and evaluation of the test shall be carried out in such a way that the real structure attains the same degree of reliability with respect to the relevant limit states and load assumptions as though design had been carried out by calculation.

General recommendation:

In determining the resistance of a structure by testing, the characteristic resistance should be defined as the lower 5% fractile determined at 75% confidence level. In determining the deformation properties of a structure, the characteristic value should be defined as the 50% fractile determined at 75% confidence level.

Boverket's handbook *Design by testing* contains descriptions regarding conditions, planning and execution, and methods for the determination of

– characteristic values and
– design values. *(BFS 1995:18)*

Partial factors for the determination of design values are given, with the exception of geostructures, in the section dealing with the specific material.

2:34 Documentation

Loadbearing structures shall be documented on drawings and in other documents in such a way that a check can be made that the requirements concerning resistance, stability and durability have been complied with.

2:4 Materials

Materials for loadbearing structures, inclusive of soil and rock, shall have known and documented properties in those respects which are significant for their use.

2:5 Design and execution *(BFS 1998:39)*

A structure shall

— be designed and executed by competent personal in a workmanlike manner in accordance with the provisions of Sections 4 – 9

— be designed so that a good standard of workmanship is made possible and so that the stipulated maintenance can be carried out

— be executed in accordance with the relevant project documentation. *(BFS 1998:39)*

During construction care shall be taken to ensure that deviations from nominal dimensions do not exceed the specified tolerances.

Deviations from construction documents shall not be made, nor action taken which is not specified in any construction document, such as holing and recessing, until it has been ascertained that the function of the element of structure will not be jeopardised. To the extent necessary, the person responsible for the design documents shall be consulted.

Temporary bracing necessary to provide stability during the period of erection shall be provided.

2:6 Supervision and control

2:61[13] Design checks

The term control of design in these mandatory provisions refers to checks on design assumptions, construction documents and calculations.

General recommendation:

This control should be undertaken by a person not previously engaged in the project. *(BFS 1998:39)*

2:62 Acceptance inspection and supervision of construction

The term acceptance inspection in these mandatory provisions refers to checks to ensure that materials and products have the specified properties, which implies that on delivery to the construction site, materials and products shall be

– identified,

– examined and

– tested unless they have received type approval or been subjected to factory production control. *(BFS 1995:18)*

The term supervision of construction in these mandatory provisions refers to checks to ensure that

– the design assumptions which were not previously verifiable and which are significant for safety are satisfied, and

– the work is carried out in accordance with the most recent drawings and other documents. *(BFS 1995:18)*

Materials and products which have received type approval or have been subjected to factory production control in accordance with Section 1:4 need not undergo further testing or inspection in those respects which are covered by the type approval or factory production control.

General recommendation:

> For products referred to in the last paragraph, acceptance inspection may be confined to identification, checks on markings and visual inspection. *(BFS 1995:18)*

:621 Basic inspection and additional inspection

The term basic inspection in these mandatory provisions refers to the general control of materials, products and workmanship. *(BFS 1995:18)*

General recommendation:

> Material-specific regulations regarding basic inspection are set out in the sections for each material.

The term additional inspection in these mandatory provisions refers to the specific control which shall be carried out regarding

– constructional details which are of critical importance for the resistance, stability or durability of the structure,

– structural details of unconventional design, and

– environmental effects. *(BFS 1995:18)*

A scheme shall be drawn up for additional inspection.

[42] Latest wording BFS 1995:18.

(BFS 1995:18)

General recommendation:

Material-specific regulations regarding additional inspection are set out in the sections for each material.

2:63 Documentation

The results of checks and supervision, inclusive of deviations if any and action taken because of these, as well as other information which is significant for the quality of the finished structure, shall be documented.

3 ACTIONS

The values of loads and actions set out in this section shall be applied in design in accordance with the method of partial factors.

Loads are to be assumed to produce static action unless it is particularly specified that their action is dynamic.

3:1 The self weight of elements of structure

The self weight of elements of structure shall be assumed to be permanent and fixed load. The weight of elements of structure which can be easily removed, relocated or added to shall be assumed to be variable free load ($\psi = 1$).

General recommendation:

The load due to non-loadbearing walls is not included in the imposed load in Section 3:4.

3:2[42] Earth load and earth pressure

The weight of earth shall be assumed to produce both a vertical load, earth load, and a horizontal or almost horizontal pressure, earth pressure. Earth load and earth pressure due to the self weight of earth shall be assumed to be permanent and fixed action. The following exceptions shall however apply:

– If it may be assumed that a certain volume of earth will be removed, its effect shall be assumed to be a variable free load, with $\gamma_f = 1.0$ and $\psi = 1$.
– The effect of temporary earthworks shall be considered in each individual case and be classified in view of the nature of the work and the compensatory actions taken.
– The effect of an action on the surface of the ground shall be classified in the same way as the action itself. *(BFS 1998:39)*

Earth load shall be calculated on the basis of the weight of the earth, consideration being given to the groundwater level, deposition of earth fill or removal of earth strata by excavation.

Earth pressure shall be calculated with regard to the nature of the soil, the groundwater level, the design, stiffness and ability to move of the supporting structure, and other influencing factors in accordance with the provisions of Section 4.

General recommendation:

Unless higher loads are specified, it should be assumed that the action on the ground surface adjacent to the structure consists of not less than q_k = 2 kN/m^2 distributed load (ψ = 1) or, where this is applicable, of a vehicle load in accordance with Subsection 3:43.

It should be noted that earth pressure may also, in addition to the self weight of the soil material and action on the ground surface, be caused by frost heave, bulking and compaction.

3:3[43] Water pressure

The water levels in lakes and groundwater which determine water pressure shall normally be determined on the basis of observations at the site.

General recommendation:

If observations are not available for the structure concerned, design can be based on nearby observation points and on a cautious assessment.

Water pressure shall be divided into two components, one of which shall be regarded as permanent action and the other as variable action.

The following shall be regarded as permanent action:

– the water pressure at mean water level in watercourses and lakes, or
– the water pressure at the mean level of the groundwater.

The difference between the water pressure at the prevailing water level and the permanent water pressure shall be regarded as a variable component. If these values are based on measured water levels such as *highest high water* HHW or *lowest low water* LLW, these shall with regard to the length of the measurement period be converted into characteristic values in accordance with Subsection 2:21.

General recommendation:

The characteristic value of action q_k should be selected so that it corresponds to HHW or LLW, and the frequent value of action ψq_k so that it corresponds to mean high water MHW or mean low water MLW.

In the analytical model, the design value q_d should be limited to what is physically possible. *(BFS 1998:39)*

The effect of a temporary lowering of the groundwater level shall be considered in each individual case and be classified in

[43] Latest wording BFS 1995:18.

view of the nature of the work and the compensatory actions taken. *(BFS 1998:39)*

Water pressure shall as a rule be assumed to be static action. Static water pressure shall be regarded as fixed action.

Dynamic forces due to rapid changes in water pressure or to wave action shall be wholly or in part regarded as free action.

3:4 Imposed load

Imposed load shall be assumed to be variable load.

3:41[44] Load due to fittings, fixtures and persons

Vertical load due to fittings, fixtures and persons shall be assumed to consist of a distributed load q_k or a concentrated load Q_k. The concentrated loads need not be combined with other variable actions. The distributed load shall be assumed to consist of two components, one fixed and the other free. When combining load due to fittings, fixtures and persons, both the fixed and the free part of the load shall be parts of one and the same load. *(BFS 1998:39)*

The number of free load components of frequent value may in a load combination, e.g. in calculating cumulative loads on columns in multi-storey buildings, be limited to three. The number of crowd loads shall not however be limited.

If the load on a storey is dependent in time and space on the imposed load on other storeys, the value of the load reduction factor ψ shall be increased. This may be appropriate for types of premises assigned to load groups 2, 3 and 4, e.g. in buildings containing several places of assembly which are often used simultaneously.

The loads set out in Table (a) overleaf relate to normal fittings and fixtures. Loads due to special equipment such as safes, archived documents or water beds, and also loads due to goods and similar, must be taken into account separately.

General recommendation:

The load due to archived documents and safes in offices may be regarded as crowd loading.

The values of free loads set out in Table (a) overleaf apply for load combinations in which the area loaded by free load is not greater than 15 m^2 in load group 1 and not greater than 30 m^2 in load groups 2 and 3.

General recommendation:

If the area loaded by free load is greater than the above values of 15 and 30 m^2, the specified load values for load groups 1, 2 and 3 (including the fixed component) can be reduced as follows. The load values shall be assumed to decrease linearly and to be equal

[44] Latest wording BFS 1995:18.

to 0.7 times the tabulated values when the loaded area is three times as large as those specified above.

Balconies, terraces and roof terraces shall, simultaneously with the distributed load according to Table (a) Item 5.1, be assumed to be acted upon by a free line load q_k = 2 kN/m (ψ = 0.5) placed 0.2 m inside the inner edge of the barrier along a side parallel to the facade. The distributed load and the line load shall be considered to be part of the same load with the same partial factor.

General recommendation:

It may be assumed that the line load is distributed over a width of 0.3 m.

Table a. Characteristic load and load reduction factor y .

Load group Type of premises/areas	Distributed load kN/m^2 Fixed load component	Distributed load kN/m^2 Free load component		Concentrated load[1] kN
	q_k (ψ = 1)	q_k	ψ	Q_k (ψ = 0)
1. *Residential loading* Rooms in residential buildings and in hotels incl. basement rooms. Patient rooms and staff rooms in institutional buildings. Attic storeys capable of conversion into accommodation.	0.5	1.5	0.33	1.5
2. *Assembly loading* Class rooms in schools, rooms in day nurseries, lecture halls. Office rooms without archives. Premises for restaurants, cafés and dining rooms, and kitchens associated with these. Laboratories. Clear spaces in libraries. Spaces with fixed seating in places of assembly such as churches, concert halls, cinemas and theatres.	1.0	1.5	0.5	3.0

General recommendation:

[1] The concentrated load Q_k = 1.5 kN should be assumed to act over a circular area of 25 mm diameter and the concentrated load Q_k = 3.0 kN over an area 100 x 100 mm in size.

Table (a) continued

Load group Type of premises/areas	Distributed load kN/m² Fixed load component	Distributed load kN/m² Free load component		Concentrated load[4] kN
	q_k (ψ = 1)	q_k	ψ	Q_k (ψ = 0)
3. *Crowd loading* Areas without fixed seating in churches, concert halls, theatres and cinemas. Museums. exhibition premises. Sales areas in department stores and shops. Gymnasiums, sports halls, dance halls. Stands with only seating. Corridors[1] in schools. Access balconies and stairs to all types of premises, except types 5:2 and 5:3.	0	4.0	0.5	3.0
4. Heavy loading Stands with only standing places. Premises with light industry and craft premises.	0	5.0	0.5	3.0
5. *Special loads* 5:1 Balconies, terraces, roof-terraces	0	2.0	0.5	1.5[2]

[1] For other corridors it shall be assumed that the load values are the same as for the type of premises of which the corridor forms a part.
[2] For single-dwelling houses the value of Q_k may be put equal to 1.0 kN.

General recommendation:

[3] The load assumptions for these types of premises/areas can often be simplified by selecting a load value between q_k = 2.5 (fixed load plus free load component for areas with fixed seating) and q_k = 4.0 kN/m^2 (fixed load plus free load component for areas without fixed seating) for the uniformly distributed load. The load value depends on the ratio of the area with fixed seating to the clear area.

[4] The concentrated load Q_k = 1.5 kN should be assumed to act over a circular area of 25 mm diameter and the concentrated load Q_k = 3.0 kN over an area 100 x 100 mm in size.

Table (a) continued

Load group Type of premises/areas	Distributed load kN/m^2 Fixed load component	Distributed Free load component		Concentrated load [1] kN
	q_k (ψ = 1)	q_k	ψ	Q_k (ψ = 0)
5:2 Attic space with not less than 0.6 m clear height and with fixed stairs to the attic space.	0.5	0.5	0	0.5
Attic space with not less than 0.6 m clear height and with access through a trapdoor of limited size (max 1x1 m).	0	0.5	0.5	0.5
5:3 Stairs in one and two storey residential buildings and stairs inside dwellings.	0	2.0	0.33	1.5
5:4 Basement roof slabs beneath courtyards, without vehicular traffic	0	4.0	0.5	3.0

General recommendation:

[1] The concentrated loads Q_k = 0.5 kN and 1.5 kN should be assumed to act over a circular area of 25 mm diameter and the concentrated load Q_k = 3.0 kN over an area 100 x 100 mm in size.

Roofs shall be assumed to be acted upon by a single concentrated load $Q_k = 1$ kN, ($\psi = 0$). For roofs with protection against falling through the roof, the concentrated load Q_k may be put equal to zero in the ultimate limit state. *(BFS 1998:39)*

General recommendation:

The load should be assumed to be distributed over a circular area of 50 mm diameter.

Structural elements shall be designed to withstand actions caused by people in rapid and violent motion (jumping, skipping, falling etc.). *(BFS 1998:39)*

A barrier for stairs, a balcony, terrace or similar shall be designed for a line action in accordance with Table (b). If failure of the barrier on a stand or similar may result in a large number of persons falling down, the line action q_k shall not be less than 3.0 kN/m. The action shall be assumed to act perpendicular to the longitudinal direction of the top of the barrier but in any direction. The action shall be regarded as a free action, with $\psi = 0$.

The values of action in Table (b) shall also be assumed to apply in designing external walls.

General recommendation:

The action may be assumed to act horizontally along a line 1.0 m above floor level and, at a window, along the bottom of the window.

Loadbearing walls, columns, balcony fronts and similar structures shall be assumed to be acted upon by a horizontal concentrated action Q_k of not less than 1.0 kN. ($\psi = 0$) placed in the most unfavourable location. However fronts below barriers designed for a line action q_k = 3.0 kN/m shall be assumed to be acted upon by a horizontal concentrated action Q_k of not less than 3.0 kN ($\psi = 0$) placed in the most unfavourable location. *(BFS 1998:39)*

Table b. Characteristic line action on a barrier

Characteristic free distributed action in accordance with Table(a), kN/m^2	Characteristic line action on barrier, kN/m
$Q_k \leq 2.0$ kN/m^2	0.4
$Q_k > 2.0$ kN/m^2	0.8
—	3 kN/m[1]

(BFS 1998:39)

[1] If failure of the barrier on stands or similar structures may result in a large number of people falling down. *(BFS 1998:39)*

General recommendation:

This action may be assumed to be distributed over a circular area of 100 mm diameter.

3:42 Load due to piece goods, bulk goods and silo pressure

The load due to piece goods, bulk goods and similar shall be calculated on the basis of the weights of these goods. The load shall be assumed to be a free load, subject to the limitations dictated by conditions.

The characteristic and frequent value of the load shall normally be determined in accordance with Subsection 2:21. If this is not possible, the characteristic value may be assumed to be the largest load which is permissible on a floor. The value of the load reduction factor ψ may be determined in each individual case.

Silo pressure shall be determined in view of the physical properties and method of handling of the fill, the design of the silo and the most dangerous storage height.

3:43 Load due to vehicles, materials handling equipment and machinery

:431[45] Load due to vehicles

Vehicles shall be assumed to produce variable free loads.

Cars in garages and multi-storey garages shall be assumed to produce a uniformly distributed vertical load of q_k = 2.0 kN/m^2 (ψ = 1.0) and a concentrated vertical load of Q_k = 10 kN (ψ = 1.0) acting on an area of 100x100 mm. These loads need not be assumed to act simultaneously. Columns, walls and similar structures shall be assumed to be subject to a concentrated horizontal action of Q_k = 5 kN (ψ = 0).

General recommendation:

> Unless some other value is shown to be more correct, the action may be assumed to act over an area of 250x250 mm within the range 0.5–1.0 m above floor level.

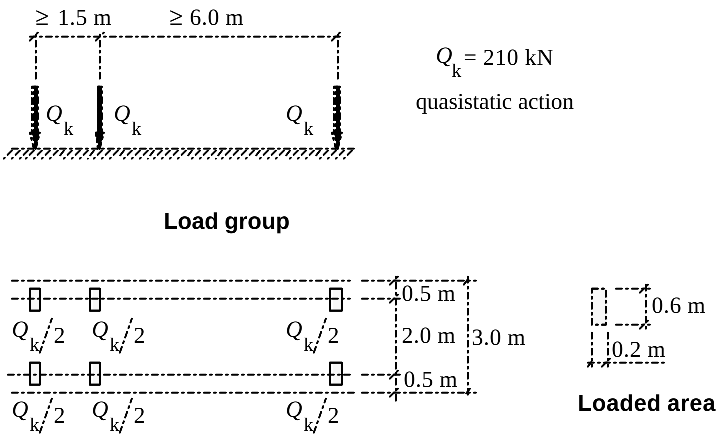
Buildings which are likely to be entered by single loaded heavy vehicles in public road or street traffic, e.g. for loading or unloading, shall be designed for a single load group (ψ = 0) in accordance with Figure (a). The load group shall be placed in the most unfavourable way within the area over which the vehicle can drive. The effect of a braking force of Q_k = 100 kN in the longitudinal direction of the load group shall also be taken into consideration.

Floor slabs in garages for large vehicles such as buses and public cleansing vehicles shall be designed for the load due to the heaviest type of vehicle which is likely to use the garage in view of the total space in the garage. For this load the load reduction factor c shall be put equal to 1.0.

Floor slabs in courtyards which are likely to carry only emergency vehicles, small lorries or maintenance vehicles shall be designed for 40% of one load group (ψ = 0) in accordance with Figure (a) and for the effect of a braking force of Q_k = 50 kN. The placing of this load group and braking force shall be as set out above for single loaded heavy vehicles in public road or street traffic.

[45] The amendment means inter alia that the last paragraph in the regulation is deleted and a new General recommendation is introduced at the end of the section.

Figure a. Prescribed action due to a vehicle



If special vehicles of a design dictated by their activity use a building, e.g. a bus and cargo terminal, fire station or aircraft hangar, the loadbearing elements of structure shall be designed for both the wheel pressures of the vehicle and a quasi-static action, i.e. the total load increased by an increment due to dynamic effects. These loads shall be determined in view of the kind of vehicle and the nature of the traffic bearing surface, e.g. surface regularity. The value of the load reduction factor ψ shall normally be put equal to 1.0.

General recommendation:

A lower value of the load reduction factor ψ may be used for special vehicles if this is warranted by the type of activity. The increment due to dynamic effects should in such a case be assumed to be not less than 25% unless it is shown by a special investigation that a lower value is warranted.

Columns, walls and similar structures, which can be exposed to collision forces shall be design for a concentrated horizontal load not less than Q_k = 5 kN (ψ = 0). *(BFS 1998:39)*

General recommendation:

Buildings which may be exposed to loads due to vehicles in public road or street traffic should be designed for actions in accordance with BRO 94, 21.22. *(BFS 1998:39)*

:432 Load due to cranes, overhead travelling cranes and similar

Cranes, overhead travelling cranes and similar shall be assumed to give rise to vertical and horizontal loads.

General recommendation:

On the basis of the crane manufacturer's load data, the load values calculated in accordance with the Crane and Lift Standardisation Association (IKH) Publication 4.30.01, *Code for steel structures for cranes*, drawn up by the Crane and Lift Commission of the Swedish Academy of Engineering Sciences (IVA), may be used unless other values can be shown to be more correct. The data in the code may be used for crane girders if these comply with the specifications regarding dimensional deviations and deformations laid down in Swedish Standard SS 764 30 05, *Cranes, Overhead Travelling Cranes and Portal Cranes – Tolerances for Cranes and Crane Girders.*

The load values in IKH 4.30.01 are characteristic values. The requirement in SS 764 30 05 regarding limitation of deformations is related to the characteristic value.

The principal forces in accordance with IKH 4.30.01 – with the exception of Duty Classes B1 and B2 – are to be regarded as fatigue actions in accordance with Subsection 2:21. The values in Table (a) below are typical values and relate to cranes of 20 years' service life. For any other service life the design stress cycle number is to be converted by linear interpolation between values of n_t. Incremental forces and special incremental forces – e.g. unbalanced forces and buffer forces – are not to be regarded as fatigue actions. For these forces it may be assumed that ψ=0. Forces due to one and the same crane which according to IKH 4.30.01 are assumed to act simultaneously are to be regarded as one single variable action. Actions due to cranes working in tandem are to be regarded as one and the same action combination.

Structures acted upon by fatigue actions from several cranes may be designed for action effects due to the crane which is most unfavourable from the standpoint of action, increased by 10%.

The values of the stress cycle number set out in Table (a) relate to the number of crane passages. In design for the local effect of wheel pressure, the stress cycle number is to be multiplied by the number of wheels.

Table a. Stress cycle number, load spectrum parameter and load reduction factor y

Duty class according to IKH 4.30.01	Design stress cycle number	Load spectrum parameter *k*	Load reduction factor ψ	
	n_t		Vertical action	Horizontal action
B1	–	–	0.5	0.2
B2	–	–	0.5	0.2
B3[1]	10^5	1/2	0.7	0.4
B4	6×10^5	1/2	0.7	0.4
B5	2×10^6	1/2	1.0	0.6
B6	2×10^6	2/3	1.0	0.6

[1] Checks for fatigue should be made in Duty Class B3 if steel S355 of Ductility Class B or Welding Quality Level WC is used. *(BFS 1998:39)*

:433 Load due to machinery and similar

Loads due to machinery and to materials or products to be found in association with machinery shall normally be assumed to be variable loads. The action due to a permanently installed part of a machine of uniquely defined and reliably determined self weight may however be assumed to be permanent.

Loads due to easily re-locatable machinery shall be regarded as free loads. The load due to a permanently installed machine may depending on circumstances be assumed to be wholly fixed or to consist of a fixed and a free load component.

In determining loads due to machinery, account shall also be taken of loads which may occur in conjunction with installation, repairs etc, for instance loads due to machine parts on the floor in the vicinity of the machine, and loads in lifting hooks.

The dynamic effect of machinery shall be taken into consideration.

General recommendation:

> The increment due to dynamic effects may without special investigation be put equal to 25% of the weight of the machine.

:434 Load due to lift machinery and similar

The structures which support lift machinery, machinery for travelators, lift pulleys, guides and similar shall be designed for the loads associated with these.

The floor of the lift machine room, including the trapdoor, shall be designed for the temporary loads which occur in conjunction with the handling and storage of lift machinery parts, but for not less than q_k = 2.0 kN/m^2 as free load ($\psi = 0$). *(BFS 1995:18)*

3:5[46] Snow load

Snow load shall be assumed to be a variable and fixed load and shall be determined as the weight per unit horizontal area.

In determining the snow load, consideration shall also be given to the effect of the shape of the building and accumulation of snow due to the action of wind, slippage and sliding.

The snow load shall be determined in accordance with formulae (a) and (b) below.

$$s_k = \mu\, C_t\, s_0 \qquad \text{(a)}$$

$$s = \psi\, s_k \qquad \text{(b)}$$

NOTATION

s_k characteristic value of snow load on a roof

μ shape coefficient which depends on the shape of the roof surface and the risk of snow accumulation due to wind, slippage and sliding

[46] Latest wording BFS 1995:18.

C_t	thermal coefficient which depends on energy losses through the roof or other thermal influence. *(BFS 1998:39)*
s_0	basic value of snow load on the ground according to Fig-ure (a)

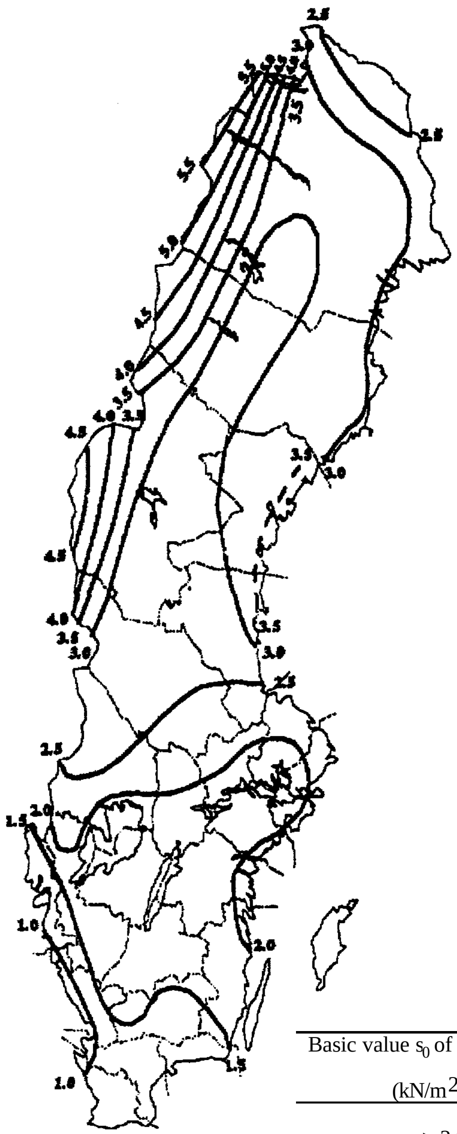
s	frequent value of snow load
ψ	load reduction factor in accordance with the table in Figure (a)

General recommendation:

The value of the thermal coefficient C_t is normally = 1.0. For roofs with little thermal insulation capacity above permanently heated spaces, the value of C_t may be put at less than 1.0.

Examples of appropriate shape coefficients and the thermal coefficient C_t, and of s_0 for all municipalities in Sweden, are given in Boverket's handbook *Snow load and wind action*.

Figure a. Prescribed snow load s_0 on the ground



Basic value s_0 of snow load (kN/m^2)	Load reduction factor ψ
$\geq$ 3.0	0.8
2.5	0.7
2.0	0.7
1.5	0.7
1.0	0.6

3:6[47] Wind action

3:61 *This heading has been withdrawn by Statute (BFS 1998:39)*

Wind action shall be assumed to be a variable action and may be regarded as a fixed action within the framework of variations given for the different shape coefficients. Exceptions can be made for structures with large sideways dimensions and if the distribution of the action is of great significance to the load effect. *(BFS 1998:39)*

General recommendation:

A third of the total wind load may in such cases be regarded as a movable action. *(BFS 1998:39)*

In calculating wind action it may be assumed that wind direction is horizontal but otherwise arbitrary.

For objects with low damping ratio and stiffness, the dynamic effects of the wind action shall be taken into account.

(BFS 1998:39)

The characteristic value W_k of wind action shall be determined from formulae (a) and (b) below.

$$W_k = m\, q_k\, A \qquad \text{(a)}$$

$$q_k = C_{dyn}\, C_{exp}\, q_{ref} \qquad \text{(b)}$$

NOTATION

μ non-dimensional coefficient whose value depends on wind direction and the shape of the elements of structure and objects acted upon by the wind

q_k characteristic value of the dynamic wind pressure

A area of the surface acted upon by wind

C_{dyn} gust coefficient which is defined in Boverket's handbook *Snow load and wind action*. For structures with high damping ratio and stiffness, the value of C_{dyn} only depends on the height h of the building and the roughness parameter of the terrain z_0. For structures with low dam-

[47] Latest wording BFS 1995:18. The amendment means inter alia that the heading 3:61, Wind action on a structure of large inherent damping and rigidity, is deleted, that the general recommendations are deleted, a new first general recommendation is added, and that text from the earlier last general recommendation is moved to a new second general recommendation.

ping ratio and stiffness, the value of C_{dyn} also depends on the dynamic properties of the structure.
(BFS 1998:39)

C_{exp} exposure coefficient whose value depends on the height z above ground of the point on, or the area of, the building for which the wind action is to be determined, the terrain parameter β and the roughness parameter z_0 of the terrain, as defined in Boverket's handbook *Snow load and wind action*

q_{ref} reference dynamic pressure due to the reference wind velocity v_{ref}, i.e. 0.5 ρ n^2_{ref}

The value of the reference wind velocity, as seen from Figure (a) overleaf, is based on a mean wind velocity during 10 minutes for terrain type II at a height of 10 m, with a return period of 50 years.

General recommendation:

Examples of suitable wind coefficients and methods for determining the wind action are given in Boverket's handbook *Snow load and wind action. (BFS 1998:39)*

The characteristic value of the component of wind action parallel to a surface may be determined from formula (a) if the shape coefficient μ is replaced by the coefficient μ_t.

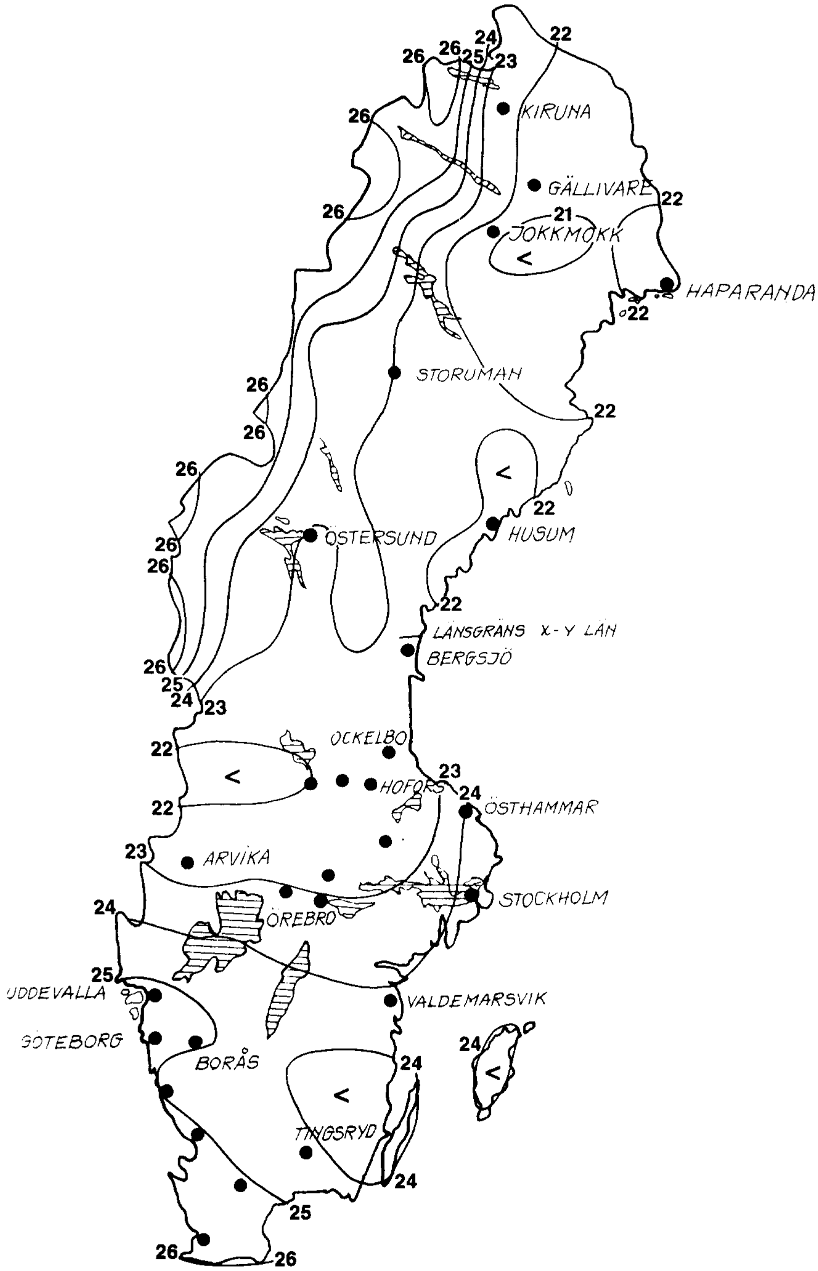
The characteristic value of the total wind action acting upon elements of structure such as ceilings and partitions, and upon objects such as unclad structural members and lattice masts, may be determined from formula (a) if the shape coefficient m is replaced by the coefficient μ_{tot}.

The frequent value of wind action shall be determined from formula (c) below.

$$W = \psi W_k \qquad \text{(c)}$$

with W_k according to formula (a) and the load reduction factor $\psi = 0.25$.

Figure a. Reference wind velocity v_{ref}, m/s, i.e. mean wind velocity during a period of 10 minutes at a height of 10 m above ground level of roughness parameter z_0 = 0.05 and with a return period of 50 years.



3:62 *has been withdrawn by Statute (BFS 1998:39)*

3:7 Pressure due to ice and water currents

Consideration shall be given to pressure due to ice and water currents.

General recommendation:

Ice pressure may be caused by changes in temperature in a stationary sheet of ice, by the pressure of current on a stationary sheet of ice, or by ice in motion. The action due to ice pressure is dependent on local conditions, the design of the structure and the properties of the ice. Changes in water level may give rise to vertical uplift and to downward drag.

Ice pressure shall be assumed to act at the level of the water surface. Ice pressure shall be assumed to be a free, variable static action which may in certain cases give rise to dy.

The action due to ice pressure need not be assumed to be divided into part actions.

3:8 Deformation action and accidental actions

Consideration shall be given to deformation action and accidental action. *(BFS 1995:18)*

General recommendation:

Appropriate methods for the determination of deformation action and accidental action are given in Boverket's handbook *Vibration, induced, deformation and accidental actions.*
(BFS 1995:18)

4 GEOSTRUCTURES

4:1 Requirements

General recommendation:

General requirements are set out in Section 2:1.

Geostructures shall be designed so that
– they do not give rise to changes in soil and groundwater conditions such that damage is caused to nearby buildings and other construction works,
– they do not unduly obstruct the planned use of adjacent ground, and
– they are not damaged by frost heave or movements caused by filling, excavation, groundwater lowering, erosion or vegetation.
Geostructures shall be designed with regard to the interaction between the structure and the subsoil.

General recommendation:

The redistribution of forces which may occur as a consequence of the rigidity of the structure and the stiffness of the subsoil should be taken into consideration.

4:2 Design assumptions

General recommendation:

General design assumptions are given in Section 2:2.

4:21 Geotechnical categories (GK)

Geostructures shall be designed, constructed and inspected in one of the geotechnical categories GK1, GK2 or GK3. GK1 shall not be applied for geostructures in Safety Class 3. Verification in accordance with Section 2:3, for geostructures in GK3, shall have a scope and quality such that the standard at no time falls below that corresponding to GK2. There are no detailed rules for GK3.

General recommendation:

GK1 and GK2 may be selected if the requirements of the mandatory provision regarding geostructures in Safety Class 3 and the conditions set out in Table (a) overleaf are satisfied. If the conditions for GK2 are not satisfied, GK3 should be applied. Well documented local experience can also be taken into consideration in selecting the geotechnical category.

Table a. Conditions for application of Geotechnical Category 1 (GK1) and Geotechnical Category 2 (GK2)

Factor	Geotechnical Category 1 (GK1)	Geotechnical Category 2 (GK2)
Soil, rock and groundwater conditions	The subsoil should, if it is subjected to a load exceeding 5 kPa, consist of soil or rock of low compressibility. The pore water pressure is lower than that corresponding to a water table level with the bottom of the excavation, unless extensive local experience shows that necessary excavation below groundwater level can be carried out without risk.	The subsoil is such that the properties of the soil and rock can be determined by well documented and generally accepted methods. The pore water pressure is lower than that corresponding to a water table situated not more than 1.0 m above the bottom of the excavation, or out without risk.in the excavation.
Geostructure	Small, conventional and relatively simple geostructure.	There is general practical experience available regarding the geostructure. Design and construction in accordance with generally accepted methods.
Conditions in the surroundings	There is no risk of collapse or slide. Nearby structures and installations are situated at such a distance that the geostructure does not affect their stability or deformations.	Conditions in the surroundings are such that they do not substantially magnify the consequences of the failure of, or deformation in, the geostructure.

General recommendation:

Examples of geostructures for which GK1 can be applied:

– Foundation structure for a building subject to normal requirements concerning limitation of the magnitude and uniformity of settlement. The design load in the ultimate limit state does not exceed 250 kN due to single columns or 100 kN/m due to a wall or several adjacent columns. The inclination to the vertical of the load resultant does not exceed 5°. The thickness of the layer of fill below the foundation structure is not greater than 1 m and it consists of compacted free draining friction soil. Piles are unspliced, precast, driven and mainly end bearing.

– Retaining structures including basement walls for which the difference in the height of backfill on each side of the structure does not exceed 2 m, and where the backfill is not compacted with an implement heavier than a 100 kg vibrating plate.

– Fill whose thickness is less than 3 m.

– Excavations above groundwater level whose depth in silt or loose cohesive soil is less than 1.5 m and in firm soil less than 3 m.

Examples of geostructures for which GK2 can be applied:

– Foundation structures for which the design vertical load in the ultimate limit state does not exceed 5 MN due to a single column or 1 MN/m due to a wall or several adjacent columns, and the mean value of the design vertical movement in the serviceability limit states is less than 0.05 m.

– Geostructures which involve excavation not exceeding 1.5 m depth in silt, 3.0 m depth in clay and 5.0 m depth in friction soil.

– Pile foundations constructed by well documented and generally accepted methods.

4:22[48] Geotechnical investigation

A geotechnical investigation shall be carried out for all loadbearing geostructures. The investigation shall elucidate the geotechnical conditions relating to the design and construction of the geostructure. The degree of detail of the investigation shall be chosen in view of the geotechnical category of the structure.

Available information concerning soil, rock and groundwater conditions and information concerning the foundations of affected buildings shall be compiled.

Further geotechnical investigations for geostructures in GK 1 are not necessary if the following requirements are satisfied:

– Available information provides the basis for a reliable assessment of ground and groundwater conditions in the area.

– An inspection of the area concerned confirms that there is no likelihood of the occurrence of any loose and compressible soil strata. The inspection shall be carried out by a geotechnician.

(BFS 1998:39)

Geotechnical field and laboratory investigations shall, in GK2 and GK3, have a scope such that information is obtained of soil, rock and groundwater conditions in those respects which are significant for the safety, function and environmental effect of the geostructure.

General recommendation:

Apart from sufficient information for the safe and economic design, construction and inspection of loadbearing structures, the geotechnical investigations should provide the information necessary for the design of drainage, frost insulation and action to prevent hygienic inconvenience due to radon gas or other substances which may emanate from the ground.

The information should be interpreted and compiled, so that the documentation on which design is based contains the following information:

– topography of the ground surface
– sequence of soil strata (materials, boundaries between strata),
– groundwater conditions,
– the material properties of soil and rock,
– the design, position and condition of adjacent structures.

[48] Latest wording BFS 1995:18.

The results of the geotechnical field and laboratory investigations should be documented in a separate report which contains no interpretations, calculations or recommendations.The documentation should be in the form of plans and sections and tables and diagrams, using the notation laid down by the Swedish Geotechnical Association. Information should be given concerning methods nd equipment used, time of year, weather, the person in charge of the investigation, the project, client etc which may be useful in interpreting the results.

In GK3, the geotechnical survey shall also comprise the investigations necessary in view of the special conditions which have resulted in the geostructure being assigned to GK3.

4:23 Characteristic values

The characteristic value of a material property shall normally be determined as its mean value. Systematic differences between the property during the investigation and in the real structure (design situation), the temporal variation of the property and errors in determining parameters, shall be taken into consideration.

The characteristic value and its dependence on depth shall be determined for each individual stratum in a soil profile. The sequence of strata in the vertical direction and the horizontal extent of the soil stratum shall be selected so that each stratum has a homogeneous composition and the same geological history.

The characteristic value of a material property may also be determined by careful choice on the basis of documented experience. This shall be systematised and formulated as a relationship between the sought property and, for instance, the results of a sounding or the value of a consistency parameter or similar.

4:24 Tolerances

For geostructures, tolerances shall be specified for dimensions which are of essential significance for the function of the structure.

General recommendation:

Examples of dimensions for which tolerances should be specified:

- the level of the surface of a fill and the level of the bottom of an excavation,
- the inclination of excavation slopes,
- the position and rake of piles,
- distance from a nearby building, other construction works or some other load.

4:25 Durability

The changes in the properties of the soil and rock material which may be expected to occur during the service life of the geostructure shall be taken into consideration in selecting the design assumptions and material parameters.

4 :3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions concerning design are set out in Section 2:3. *(BFS 1995:18)*

4:31 Design in the ultimate limit states

In the case of geostructures, special consideration shall be given, in addition to the conditions in Section 2, to an ultimate limit state which is characterised by the fact that movements of the geostructure cause material failure or loss of support for part of the supported loadbearing structure or an adjacent structure even if the bearing resistance of the soil is not exceeded.

General recommendation:

A method for calculating deformations should take account of a possibly nonlinear relationship between load and deformation.

Design in the ultimate limit states, in cases where the provisions of Subsections 4:31, 4:32 and 4:33 are not applicable, shall be carried out in accordance with the principles in Sections 2:3 and 4:2. *(BFS 1995:18)*

The value of the partial factor γ_m shall be selected on the basis of the conditions set out in Table (a) below

Table a. Conditions to be taken into consideration in selecting the value of the partial factor γ_m in the ultimate limit states.

Favourable conditions	Unfavourable conditions
Experience shows that the material property has little scatter.	Experience shows that the material property has large scatter.
Test results from the geotechnical investigation show that scatter is normal.	Test results from the geotechnical investigation show that scatter is greater than normal.
The investigations are of large scale and permit good determination of the material property.	The investigations are of small scale.
The investigations are carried out by well documented methods which give reproducible results.	The investigations are carried out by methods which have poor reproducibility or by methods based on limited experience.
Additional checks on the material property.	No additional checks on the material property.
Little uncertainty in converting test results into the sought results material property.	Great uncertainty in converting test results into the sought material property.
Failure is ductile.	Failure is brittle.

(BFS 1995:18)

General recommendations:

In designing geostructures, γ_m should take account of the uncertainty in the determined value of the material property, and γ_{Rd} of the uncertainty in the analytical model and the design assumptions.

The value of γ_m should be chosen on the basis of table (b) below and in such a way that the lower limiting value is taken only if conditions are in all respects favourable, and the upper limiting value if unfavourable conditions predominate. In other cases a reasonable intermediate value of γ_m is to be chosen. If conditions are unfavourable in many respects, the geotechnical investigations should be extended.

Table b. Partial factor g_m in the ultimate limit states.

Material property	Partial factor γ_m
Modulus and other deformation parameters	1.2 – 1.8
Strength parameter tan φ	1.1 – 1.3
Other strength parameters	1.6 – 2.0

If the mode of action or extent of the geostructure is such that the bearing resistance is not determined by a local value of the material property, the value of γ_m can be reduced. The value of γ_m can be reduced by 20% in cases where the bearing resistance of the geostructure is determined by the mean value of the material property. If the bearing resistance is to some extent determined by a local value of the material property, a reasonable reduction between 0 and 20% can be made. However, in determining the value of-tan φ_d a value lower than γ_m = 1.05 should not be used. For other material properties the value of the partial factor γ_m should not be lower than 1.0.

In designing for accidental actions, the value of the partial factor γ_m can be reduced by 10%. A value lower than γ_m = 1.0 should not however be used.

When a high value of a material property is unfavourable for the geostructure, e.g. in determining the action effect of an imposed deformation, the design value should be selected so that it is not less than the 95% fractile of the property.

:311 Earth pressure

:3111 Geotechnical category 1 (

General recommendation:

General provisions concerning earth pressure are set out in Section 3:2.

Earth pressure due to uncompacted backfill shall for drained and flexible structures be calculated according to formula (a) below.

$$p_d = k_d(g_d z+q_d) \qquad (a)$$

NOTATION

p_d design intensity of earth pressure at depth z below the ground surface

k_d design earth pressure coefficient for backfill. The value of k_d shall be taken to be

0.35 for sand and gravel
0.5 for silt
0.6 for clay

g_d unit weight of backfill (design value)

z depth below ground surface

q_d external uniformly distributed design load on the ground surface (see Section 3:2), situated at a distance from the structure less than 1.5 × the foundation depth.

For rigid structures earth pressure shall be assumed to be 50% higher.

Earth pressure due to compaction shall be given special consideration as appropriate.

General recommendation:

The structure should be designed so that earth pressure does not increase in winter as a result of frost action.

:312 Foundation slabs

:3121[49] Geotechnical Category 1 (GK1)

The requirements in both the ultimate and serviceability limit states shall be considered to have been satisfied for foundation slabs in GK1 if

– the action resultant does not deviate more than 5° from the vertical,
– the width and foundation depth of the foundation slab are each not less than 0.4 m, and
– condition (a) below is satisfied.

The requirement concerning foundation depth does not however apply on rock.

$$S_{vd} \leq R_{vd} \tag{a}$$

[49] Latest wording BFS 1995:18. The amendment means that the general recommendation is deleted.

NOTATION

S_{vd} vertical design load in the ultimate limit states including self weight and backfill if any on the structure

R_{vd} design vertical bearing resistance, $f_d A_{ef}$

f_d design foundation pressure according to Table (a) below

A_{ef} effective foundation area $b_{ef} \times l_{ef}$ according to Figure a.

Figure a. Figure for calculation of effective foundation area

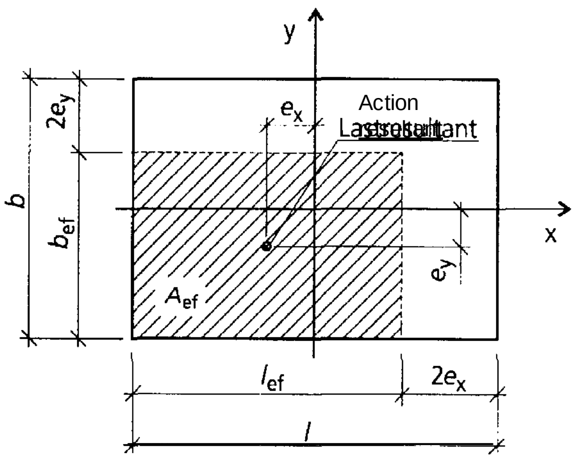


Table a. Design values of foundation pressure for slabs in Geotechnical Category 1 (GK 1)

Material	f_d (kPa)	Material	f_d (kPa)
Rock (unweathered)	400	Sand	100
Till	200	Silt	50
Gravel	150	Firm clay[1]	100

[1] Characteristic shear strength > 50 kPa under undrained conditions

For sand and silt the value of f_d shall be limited to one half the tabulated value if the groundwater level is higher than one slab width below the foundation level.

If different soil strata occur within a depth of twice the slab width measured from the foundation level, the design value of foundation pressure shall be determined on the basis of the worst material in question.

:3122 Geotechnical Category 2 (GK2) and Geotechnical Category 3 (GK3)

The bearing resistance of foundation slabs shall satisfy condition (a) below.

$$\boldsymbol{S}_{\mathrm{d}} \leq R_{\mathrm{d}} \qquad \text{(a)}$$

NOTATION

S_d design action effect in the ultimate limit states

R_d design bearing resistance in the ultimate limit states

General recommendation:

General provisions regarding tilting, uplift and sliding are given in Subsection 2:112.

:313 Piles

:3131 Geotechnical Category 1 (GK1)

A foundation structure in GK1 shall be designed so that the load resultant does not deviate by more than 5° from the vertical. The requirements in the ultimate and serviceability limit states shall in GK1 be considered to have been complied with for precast unspliced piles which

– are designed in accordance with Swedish Standard SS 81 11 03,
– have a design load effect ≤ 300 kN per pile,
– trasmit most of the load to rock or loadbearing soil stratum at the pile point, and
– are driven to refusal in accordance with the following rules.

Driving to refusal shall be carried out with a free fall hammer or a single line drop hammer. The drop for a free fall hammer shall be 0.3 m and for a single line drop hammer 0.4 m. The hammer shall weigh not less than 3 tonnes.

Driving to refusal shall comprise not less than three series of ten blows, during which the set shall be constant or decreasing, and not more than 10 mm per 10 blows. Before piling is terminated, redriving shall be carried out.

General recommendation:

Redriving should comprise not less than 25% of the number of piles. If the set during redriving exceeds the value specified above, all piles should be redriven.

If the pile point reaches rock or supposed rock, the pile shall be embedded in this before driving to refusal is carried out. After embedment in the rock, driving to refusal may be carried out by one series of 10 blows if the set due to this does not exceed 3 mm.

:3132 Geotechnical Category 2 (GK 2) and Geotechnical Category (G

The bearing capacity of all the parts of the precast pile shall be determined in accordance with the design regulations for the material concerned. Verification of the bearing resistance of piles with regard to failure of the surrounding soil or rock shall be carried out either by calculation or by testing, due consideration being given to the prevailing soil and driving conditions.

General recommendation:

For piles which are substantially end bearing piles, bearing resistance can be verified by driving to refusal.

The maximum value of the permanent set during driving to refusal should be selected within the range 10 – 30 mm per 10 blows.

In GK2 the conditions governing driving to refusal can be determined on the basis of well documented and systematised experience.

If the conditions governing driving to refusal are determined by test piling and dynamic load tests in accordance with Subsection 4:33, the test should comprise not less than 5% of the number of piles, but not less than four, distributed over the area.

In cases where piling warrants construction in GK3, the conditions governing driving to refusal shall be determined with regard to the structure in question if bearing resistance is verified in this way.

4:32 Design in the serviceability limit states

It shall be borne in mind that for geostructures the serviceability limit states often constitute the design criterion.

General recommendation:

In designing geostructures in the serviceability limit states, the interaction between soil (rock) and the other parts of the structure should be taken into consideration. Limiting values *C*, for instance for deformations, should be laid down so that, in addition to the requirements in Subsection 2:12, the requirements of the client regarding limitation of maintenance and service costs are also taken into consideration.

In calculating deformations, it should be borne in mind that the load-deformation curve for geostructures is often nonlinear.

For calculating deformations due to actions of long duration, the actions can be determined in accordance with Subsection 2:21. Appropriate values of ψ_1 are set out in *BBK 94*, Subclause 2.2.2.

The design criterion for settlement in the serviceability limit states can be put at

$$s_d \le s_{acc} \quad \text{(a)}$$

$$\Delta s_d \le \Delta s_{acc} \quad \text{(b)}$$

NOTATION

s_{acc}	acceptable total settlement
Δs_{acc}	acceptable differential settlement

Design values of the settlement s_d or the differential settlement Δs_d can be determined in different ways:

– The settlements are calculated with the design values of the parameters according to Subsections 2:322 and 4:31. The uncertainty in the analytical model and the design assumptions is to be taken into consideration by γ_{Rd}. The partial factor γ_n is put equal to 1.0, and the value of γ_m is put at 90% of the corresponding factor in the ultimate limit states, but not less than 1.0.

– The settlements are calculated with the characteristic values of the parameters according to Subsection 4:23 after which the total settlement or differential settlement is corrected with respect to the uncertainty in the calculated value either by adding an increment or multiplying it by a factor based on compiled and evaluated experience data.

4:33 Design by testing *(BFS 1995:18)*

The bearing resistance and deformations of geostructures may be determined on the basis of the results of tests. In such a case, primary consideration shall be given to the following factors:

– Deviations in soil and groundwater conditions between the test site and the site of the planned geostructure.

– Temporal effects.

– Scale effects.

– Differences in the mode of action during the test and during design.

General recommendation:

Characteristic values of bearing resistance and deformation properties can, if a sufficient number of tests are made, be determined in accordance with Subsection 2:33 and Boverket's handbook *Design by testing*. See Subsection 4:23.

If a small number of load tests are carried out, the distribution of test results shall be used only to check that an empirical design procedure is appropriate for the planned design of the geostructure in the prevailing soil and groundwater conditions. The structure shall in the ultimate and serviceability limit states be designed in accordance with Subsections 4:31 and 4:32.

4:4 Materials

General recommendation:

General provisions regarding materials are given in Section 2:4.

4:5[50] Execution and workmanship

General recommendation:

General provisions regarding execution and workmanship are given in Section 2:5.

For the construction of geostructures in GK2 and GK3, an execution procedure sheet shall be drawn up in consultation with the designer of the geostructure. *(BFS 1998:39)*

General recommendation:

The execution procedure sheet should set out the working procedure and the sequence of operations, and restrictions with respect to e.g. harmful deformations. The execution procedure sheet should be related to the design and to the inspection.
(BFS 1998:39)

During construction a diary shall be kept to document the work carried out, precipitation, temperature and any other observations which may be useful in evaluating the structure.

4:6[51] Supervision and control

The values of the partial factor γ_m set out in this section presuppose that the supervision and control referred to in Section 2:6 is carried out.

General recommendation:

Basic inspection of geostructures should in all geotechnical categories comprise a check that the real soil, rock and groundwater conditions agree with the assumptions on which design has been based.

Additional inspection of geostructures in GK2 should comprise checks, formulated with respect to the structure in question, regarding the bearing resistance, function and durability of the structure and its effect on the surroundings.

Additional inspection of geostructures in GK3 should be carried out in accordance with the rules for GK2, subject to the following additional measures. Checks by the design team shall be supplemented by inspection carried out by an independent expert unconnected with the project in question. *(BFS 1998:39)*

[50] Latest wording BFS 1995:18.
[51] Latest wording BFS 1995:18.

5 TIMBER STRUCTURES

5:1 Requirements

General recommendation:

General requirements are set out in Section 2:1.

5:11 Durability

Timber structures shall be designed and constructed so that harmful attack by rot and wood destroying insects is prevented. Structural connectors of steel shall be protected against harmful corrosion, and glued joints shall have satisfactory durability.

5:2 Design assumptions

General recommendation:

General design assumptions are set out in Section 2:2.

5:21 Service classes

In designing timber structures, consideration shall be given to the effect of moisture on resistance and stiffness. This shall be done by assigning the elements of structure to one of the following service classes and using the appropriate factors in determining the design values.

Service Class 0 is characterised by an environment in which relative humidity exceeds 65% for only a few weeks each year and, on average, does not exceed 40%.

Service Class 1 is characterised by an environment in which relative humidity exceeds 65% for only a few weeks each year and at no time reaches 80%.

Service Class 2 is characterised by an environment in which relative humidity exceeds 80% for only a few weeks each year.

Service Class 3 is characterised by an environment which produces in the materials a higher moisture content than that corresponding to Service class 2.

5:22 The duration of actions

The effect of the duration of actions on resistance and stiffness shall be taken into consideration in designing timber structures. This shall be done by means of modification factors κ_r and κ_s in

accordance with Subsections 5:3121 and 5:322. These factors shall be determined with respect to the load duration classes set out in Table (a) below.

Table a. Classification of actions in view of their duration

Load duration Class	Cumulative duration of action	Example of action[1]
Permanent load		
Load type P	more than10 years	Self weight of permanent parts of the building.
Variable load		
Load type A	between 6 months and 10 years	The fixed portion of the imposed. load. Snow load of frequent v alue.
Load type B	between 1 week and 6 months	The free portion of the imposed load. Wind action of frequent value. Snow load of characteristic value. Actions on concrete formwork and similar temporary structures.
Load type C	less than 1 week	Wind action of characteristic value. Single concentrated load on a roof.

[1] The above examples are intended only as general recommendations.

5:23 Characteristic values for wood based materials

Unless it can be shown that other values are more appropriate, the characteristic values set out in Tables (a) – (d) below may be used in calculating the resistance and stiffness of structural timber, glued laminated timber and wood based structural grade boards (K plywood, K board, K particleboard and flooring grade particleboard). *(BFS 1998:39)*

General recommendation:

The tabulated characteristic values should in some cases be adjusted to take account of size effects.

In tension perpendicular to the grain the effect of size may be taken into account according to Subsection 5:3122.

For glued laminated timber, the effect of size in bending and tension parallel to the grain may be taken into account by multiplying f_{mk} and f_{tk} in Table (b) by the factor k_h given in formula (a)-(b) below for h less than 600 mm. *(BFS 1998:39)*

$$k_h \leq \begin{cases} 1{,}15 & \text{for } h \leq 300 \quad \text{(a)} \\ \left(\dfrac{600}{h}\right)^{0{,}2} & \text{for } 300 < h < 600 \quad \text{(b)} \end{cases}$$

where h is the depth of the beam concerned (mm). *(BFS 1998:39)*

If a number of timber members interact and it can be shown, for instance in sheet piling, that this increases strength and stiffness, characteristic values higher than those in Tables (a) and (b) may be applied. *(BFS 1998:39)*

Table a.[52] **Characteristic values (MPa) for calculation of the resistance and stiffness of structural timber**

Structural timber	K35	K30	K24	K18	K12
Strength properties					
Bending parallel to grain, f_{mk}	35	30	24	18	12
Tension parallel to grain, f_{tk}	21	20	16	11	8
Tension perpendicular to grain, f_{t90k}	0.5	0.5	0.5	0.5	0.5
Compression parallel to grain, f_{ck}	30	29	23	17	14
Compression perpendicular to grain, f_{c90k}	7	7	7	7	7
Longitudinal shear, f_{vk}[1]	3	3	3	3	3
Stiffness properties for calculation of resistance					
Modulus of elasticity E_{rk}	9 000	8 700	6 900	5 100	4 200
Shear modulus, G_{Rk}	610	600	450	350	300
Stiffness properties for calculation of deformations					
Modulus of elasticity parallel to grain, E_k	13 000	12 000	10 500	9 000	8 000
Modulus of elasticity perpendicular to grain, E_{90k}	430	400	350	300	250
Shear modulus, G_k	810	800	700	600	500

[1] The value of transverse shear may be put equal to one half the value of longitudinal shear. *(BFS 1998:39)*

[52] The amendment means inter alia that the values for glued laminated timber are moved to Table b.

Table b. Characteristic values (MPa) for calculation of the resistance and stiffness of glued laminated timber and glued structural timber

Glued laminated timber[1] Glued structural timber	L40	L30	LK30	LK20
Strength properties				
Bending parallel to grain, f_{mk}	33[3]	26[3]	30	24
Tension parallel to grain, f_{tk}	23	17	20	16
Tension perpendicular to grain, f_{t90k}	0.5	0.5	0.5	0.5
Compression parallel to grain, f_{ck}	36	29	29	23
Compression perpendicular to grain, f_{c90k}	8	7	7	7
Longitudinal shear, f_{vk}[2]	4[4]	3	3	3
Stiffness properties for calcul a-tion of resistance				
Modulus of elasticity E_{Rk}	10 400	8 700	8 700	6 900
Shear modulus, G_{Rk}	700	600	600	450
Stiffness properties for calcul a-tion of deformations				
Modulus of elasticity parallel to grain, E_k	13 000	12 000	12 000	10 500
Modulus of elasticity perpen-dicular to grain, E_{90k}	450	400	400	350
Shear modulus, G_k	850	800	800	700

(BFS 1998:39)

[1] The tabulated characteristic values for glued laminated timber refer to beam cross sections ≥ 600 mm depth.

[2] The value of transverse shear may be put equal to one half the value of longitudinal shear.

[3] In bending with the moment vector perpendicular to the plane of the glue joint, the value of f_{mk} may however be put at not more than 26 MPa for L40 and not more than 21 MPa for L30.

[4] The tabulated value refers to beams of rectangular section. For beams of non-rectangular section, f_{vk} = 3 MPa. *(BFS 1998:39)*

Table c. Characteristic values (MPa) for calculation of the resistance and stiffness of K‑plywood (structural grade plywood). *(BFS 1998:39)*

K- plywood	*Strength Class* P40	P30	P20
Strength properties			
Bending about an axis in the plane of the board, f_{mk}[1]	40	30	20
Tension parallel to the plane of the board, f_{tk}[1]	35	25	20
Tension perpendicular to the plane of the board, f_{t90k}	0.5	0.5	0.5
Compression parallel to the plane of the board, f_{ck}[1]	30	25	20
Compression perpendicular to the plane of the board f_{c90k}	7	7	7
Panel shear, f_{pk}[2]	3	3	3
Rolling shear, f_{vk}[3]	1	1	1
Stiffness properties for calculation of resistance			
Modulus of elasticity parallel to the board, E_{Rk}[1]	8 700	8 700	6 900
Shear modulus in panel shear, G_{Rk}[2]	450	450	350
Stiffness properties for calculation of deformations			
Modulus of elasticity parallel to the board, E_k[1]	12 000	12 000	10 500
Shear modulus in panel shear, G_k[2]	600	600	500

1 Only plies with the direction of the grain parallel to the direction of stress shall be considered. The values presuppose that not less than two plies are effective. If only one ply is effective, the values shall be reduced by half.

2 All plies may be considered. The tabulated values refer to shear stresses parallel to a grain. When stress is at an angle of 45° to the grain, the values may be doubled.

3 All plies may be considered.

Table d. Characteristic values (MPa) for calculation of the resistance and stiffness of K board, K particleboard and flooring grade particleboard. *(BFS 1998:39)*

	K board Strength Class				K particleboard Flooring grade Particleboard ($t \geq 19$ mm) Thickness t (mm)		
	K50	K40	K35	K13	<14	14-19	>19
Strength properties							
Bending about an axis in the plane of the board, f_{mk}	44	38	31	13.5	20	18	16
Tension parallel to the plane of the board, f_{tk}	25	22	20	6	10	9	8
Tension perpendicular to the plane of the board, f_{t90k}	0.85	0.85	0.65	0.1	0.6	0.5	0.4
Compression parallel to the plane of the board, f_{ck}	25	22	20	6	12	12	10
Compression perpendicular to the plane of the board, f_{c90k}	25	24	22	4	7	7	7
Panel shear, f_{pk}	15	14	12	3.5	7	6	5
Rolling shear, $f_{v\,k}$	2.4	2.4	1.5	0.2	2	1.6	1.3
Stiffness properties for calculation of resistance							
Modulus of elasticity, bending, E_{rk}	4 000	4 000	3 500	2 000	2 000	1 800	1 800
Tension and compression, E_{tRk}, E_{cRk}	4 000	4 000	3 500	2 000	1 800	1 500	1 500
Shear modulus, G_{Rk}	1 700	1 700	1 500	800	900	700	700
Stiffness properties for calculation of deformations							
Modulus of elasticity, bending E_k	6 700	5 000	4 500	3 000	4 000	3 500	3 000
Tension and compression, $E_{tk,}$ E_{ck}	6 700	5 000	4 500	3 000	2 600	2 400	2 200
Shear modulus, G_k	3 350	2 100	1 900	1 300	1 300	1 200	1 100

5:24 Characteristic loadcarrying capacity of timber joints

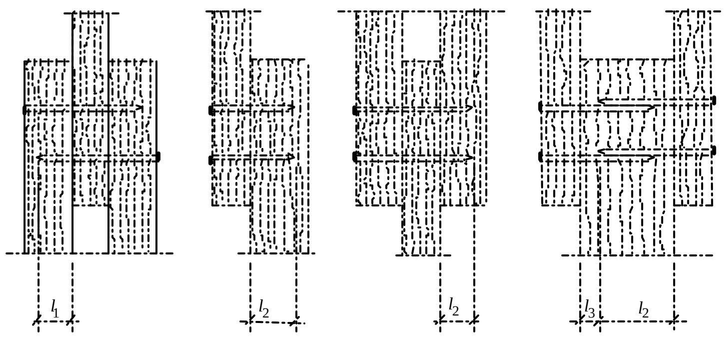
The characteristic values of the loadcarrying capacity of timber joints shall be determined with regard to those properties of the timber and the fasteners which are significant for their loadcarrying capacity and durability.

General recommendation:

Determination of characteristic values on the basis of test results should be made in accordance with Boverket's handbook *Design by testing*.

:241 Nailed joints subject to shear force

Figure a. Pointside penetration lengths for nails



General recommendation:

If the conditions in Section 5:4 are satisfied, the characteristic value of the loadcarrying capacity R_{vk} (N) of a nail per shear plane can be determined from formulae (a) – (b) below.

$R_{vk} = 150\ d^{1.7}$ for a square or grooved wire nail (a)

$R_{vk} = 125\ d^{1.7}$ for a round nail (b)

where d is the least side dimension (mm) of the nail.

The value of R_{vk} according to the formulae is valid on condition that

— the nails are driven perpendicular to the grain,

— the thickness of the thinnest piece of timber is at least 7d,

— the pointside penetration lengths of nails inclusive of the points as shown in figure (a) are

$l_1 \geq 8d$ from both sides for nails in double shear

$l_2 \geq 12d$ for smooth nails in accordance with formula (a) or (b) *(BFS 1998:39)*

$l_2 \geq 8d$ for annular ringed shank nails and helically threaded nails,

– the spacing of nails along the grain and the distance between a nail and an unloaded end of a timber is not less than $10d$, and the distance between a nail and a loaded end of a timber is not less than $15d$,

— the spacing of nails perpendicular to the grain and the distance between a nail and an unloaded edge of a timber is not less than 5*d*, and the distance between a nail and a loaded edge of a timber is not less than 10*d*, and

— the joint contains not less than 2 nails.

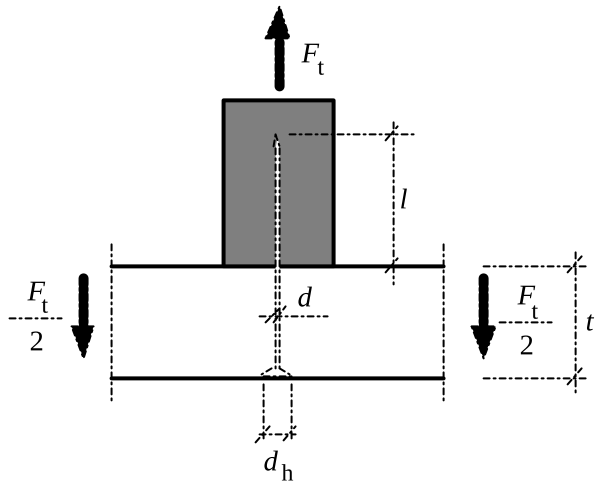
Definitions of unloaded and loaded edge of the timber, and unloaded and loaded end respectively, are given in Figure 6.3.1.2a of SS-ENV 1995-1-1. *(BFS 1998:39)*

For thinner pieces of timber or shorter pointside penetration lengths the value of R_{vk} is to be reduced for the shear plane concerned in proportion to the thickness of the thinnest piece of timber or the shortest pointside penetration length. The thickness of timber and the pointside penetration length l_1 shall not however be less than 5*d*. The pointside penetration length l_2 shall not be less than 8*d* for smooth nails nor less than 5d for annular ringed shank nails and helically threaded nails. In addition, for joints in single shear, l_2 shall be not less than half the length of the nail. If the dimension l_3 according to Figure (a) is greater than 3*d*, the nails from the side pieces may touch each other in the middle piece.

A higher characteristic value of the load-carrying capacity of a nail may in some cases be applied for nailed steel to timber joints, see for example SS-ENV 1995-1-1, 6.3.1.4, and its associated NAD(S). *(BFS 1998:39)*

:242 Nailed joints subject to withdrawal force

Figure a. Nail subject to withdrawal force



General recommendation:

If the conditions in Section 5:4 are satisfied, the characteristic value of the loadcarrying capacity of nails anchored in timber, subject to a withdrawal force, can be determined as the least value of R_{tk} according to formulae (a) – (c) below.

$$R_{tk} \le \begin{cases} d\,l\,f_{tk} & \text{(a)} \\ d\,t\,f_{tk} + d^2 f_{hk} \quad \text{only smooth nails} & \text{(b)} \\ d^2 f_{hk} \quad \text{only annular ringed shank nails and helically threaded nails} & \text{(c)} \end{cases}$$

The value of R_{tk} holds both when nails are driven perpendicular to the grain and when they are driven symmetrically on the skew (slant nailing), provided that

— the pointside penetration length l is not less than $12d$ for smooth nails and not less than $8d$ for annular ringed shank nails and helically threaded nails,

— the nails are driven at an angle of 45° or greater to the grain,

— the nails are not driven into the end grain.

The strength parameters f_{tk} and f_{hk} are to be taken from Table (a) below. For annular ringed shank nails the pointside penetration length l refers only to the annular portion of the nail.

Table a. Strength parameters f_{tk} and f_{hk} (MPa) for nails subject to a withdrawal force

	f_{tk}	f_{hk} [1]
Square and grooved nail	0.9	50
Round nail	0.7	50
Annular ringed shank nail, helically threaded nail	3	50
Hot dip zinc coated annular ringed shank nail	2	50

[1] The values presuppose that $d_h \geq 2.5d$

:243[53] Bolted joints subject to shear force

General recommendation:

For a bolt subject to shear force, the characteristic value of the loadcarrying capacity per shear plane can be determined as the least value of R_{vk} (N) according to formulae (a) – (e) below.

$$R_{vk} \leq \begin{cases} 6(\kappa_1\, t_1 + \kappa_2\, t_2)d & \text{only joint in single shear} \quad \text{(a)} \\ 12\,\kappa_2\, t_2\, d & \text{only joint in double shear} \quad \text{(b)} \\ 24\,\kappa_1\, t_1\, d & \text{(c)} \\ 4\,\kappa_1\, t_1\, d + 22\, d^2 & \text{(d)} \\ 30\, d^2 \sqrt{\kappa_1 + \kappa_2}\, \sqrt{\dfrac{f_{yk}}{240}} & \text{(e)} \end{cases}$$

NOTATION

t	thickness of a timber member (mm)
d	diameter of bolt (mm)
f_{yk}	yield stress of bolt material (Mpa)
κ	factor which takes account of the angle α between the direction of force and the direction of grain and which can be determined according to Figure (a) or formulae (g) – (h) below.
α	angle between the direction of force and the direction of grain

In a joint in double shear, the subscript 1 denotes the outside member and the subscript 2 the middle member. For a joint in single shear, the subscripts are to be chosen so that

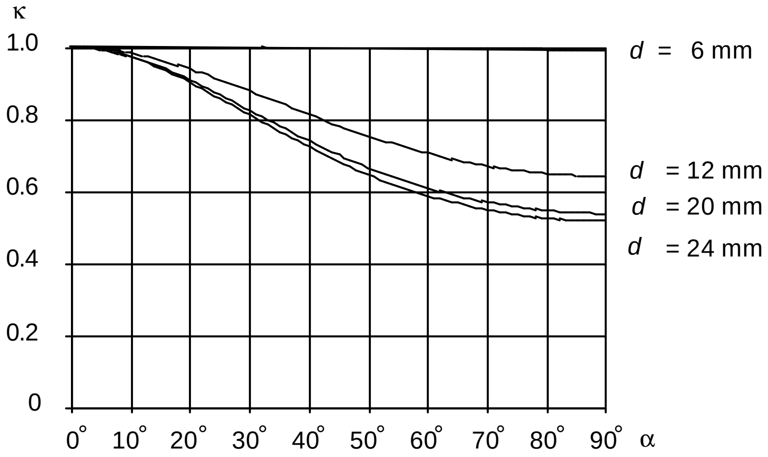
$$\kappa_1 t_1 \leq \kappa_2 t_2 \qquad \text{(f)}$$

The value of R_{vk} presupposes that the spacing of bolts along the grain and the distance between a bolt and the end of a timber member is not less than 7*d*, and the spacing of bolts perpendicular to the grain and the distance between a bolt and a loaded edge of a timber member is not less than 4*d*. The minimum distance between a bolt and an unloaded timber edge may however be 2*d*.

Higher values can normally be applied for nailed steel to timber joints.

[53] The amendment means inter alia that the last paragraph in the general recommendation before Table a is deleted.

Figure a. Factor k **in a bolted joint**



$$k = \frac{k_{90}}{k_{90}\cos^2 a + \sin^2 a} \qquad \text{(g)}$$

$$k_{90} = 0.45 + 8d^{-1.5} \qquad \text{(h)}$$

A higher characteristic value of the load-carrying capacity of a bolt may in some cases be applied for bolted steel to timber joints, see for example SS-ENV 1995-1-1, 6.5.1.4, and its associated NAD(S). *(BFS 1998:39)*

:244[54] Screwed joints subject to shear force

General recommendation:

For a wood screw inserted perpendicular to the grain, the characteristic value of the loadcarrying capacity per shear plane, under the action of shear force, can be determined as the least value of R_{vk} (N) according to formulae (a) – (c) below.

$$R_{vk} \le \begin{cases} 24\kappa_1\, t\, d & \text{(a)} \\ 4\kappa_1\, t\, d + 22d^2 & \text{(b)} \\ 26\, d^2 \sqrt{\kappa_1 + \kappa_2}\, \sqrt{\dfrac{f_{yk}}{180}} & \text{(c)} \end{cases}$$

NOTATION

t headside timber thickness (mm)

d shank diameter of the screw (mm)

κ factor which takes account of the angle a between the direction of force and the direction of grain according to Subsection 5:243

f_{yk} yield stress of the material of the screw (MPa)

[54] Latest wording BFS 1995:18.

The value of R_{vk} presupposes that the joint comprises not less than two screws and that the provisions of Subsection 5:243 regarding spacing and distances are applied. It is further presupposed that the thickness of the timber member is not less than 2*d* and that the pointside penetration length of the screw is not less than 8*d*. If the pointside penetration length is less than 8*d*, the value of R_{vk} is to be reduced in proportion to the length. The pointside penetration length shall not however be less than 5*d*.

A higher characteristic value of the load-carrying capacity of a wood screw may in some cases be applied for screwed steel to timber joints. *(BFS 1998:39)*

:245 Screwed joints subject to withdrawal force

General recommendation:

For a screw inserted perpendicular to the grain, acted upon by a withdrawal force, the characteristic value of the loadcarrying capacity, R_{tk} (N), can be determined from formula (a) below.

$$R_{tk} = 11\,(2.5 + d)\,(l_g - d) \qquad \text{(a)}$$

NOTATION

d shank diameter of screw (mm)
l_g threaded pointside penetration length (mm)

:246 Glued joints

The characteristic value of the shear strength in a glued joint shall be not greater than the lowest characteristic value of the shear strength for the materials comprised in the joint. In types of glued joints other than those between unspliced laminates and between unspliced flanges and webs in beams, the risk of nonuniform stress distribution shall be taken into consideration.

5:3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions concerning design are set out in Section 2:3. *(BFS 1995:18)*

5:31 Design in the ultimate limit states

:311 Calculation of forces and moments

General recommendation:

In calculating forces and moments in timber structures, a linear relationship between stress and strain may be assumed for the wood material.

Displacements in mechanical joints shall be taken into consideration.

Where several connectors interact in a joint, distribution of forces in the joint shall be determined with regard to the deformation of the timber members and the stiffness and deformation capacity of the connectors.

General recommendation:

The load-carrying capacity for laterally loaded bolts in line with the load direction may be determined in accordance with NAD(S) to SS-ENV 1995-1-1, 6.5.1.1. *(BFS 1998:39)*

Movements in timber structures caused by moisture shall be taken into consideration if they have significance for the resisstance.

In determining section properties, the effect due to reduction of the cross section shall be taken into consideration. Holes due to bolts, screws and nails need not however be taken into consideration if the lateral dimensions of the connectors are not greater than 6 mm.

In calculating shear forces in a beam supported at its bottom edge and loaded at the top, such load may be ignored if the distance between the load and the theoretical point of support is less than the depth of the beam.

:312 Calculation of resistance

The resistance of a timber structure shall be determined in accordance with the elastic theory unless some other procedure can be shown to be more correct. It shall however be borne in mind that the resistance of timber and wood based material may be limited by compressive strain.

In joints subject to tension perpendicular to the grain, the risk of splitting shall be taken into consideration. *(BFS 1995:18)*

:3121[55] Design values of material pro

The design values of strength, resistance, modulus of elasticity and shear modulus in the ultimate limit states shall be determined from formulae (a) – (d) below.

$$f_d = \frac{k_r\ f_k}{\gamma_m \gamma_n} \qquad \text{(a)}$$

$$R_d = \frac{k_r\ R_k}{\gamma_m \gamma_n} \qquad \text{(b)}$$

$$E_{Rd} = \frac{k_r\ E_{Rk}}{\gamma_m \gamma_n} \qquad \text{(c)}$$

$$G_{Rd} = \frac{k_r\ G_{Rk}}{\gamma_m \gamma_n} \qquad \text{(d)}$$

NOTATION

f_k, R_k	characteristic value of strength and resistance respectively in accordance with Subsections 5:23 and 5:24
E_{Rk}	characteristic value of modulus of elasticity for calculation of resistance in accordance with Subsection 5:23
G_{Rk}	characteristic value of shear modulus for calculation of resistance in accordance with Subsection 5:23
γ_m	partial factor for resistance
γ_n	partial factor for safety class in accordance with Subsection 2:115
κ_r	modification factor which takes account of the effect of moisture and load duration in accordance with Subsections 5:21 and 5:22. Values of κ_r are set out in Tables (a) – (e) below.

In the ultimate limit states the value of the partial factor γ_m shall be put equal to 1.25. In type approved structures and structures subject to factory production control, where design and production are carried out in such a way that a smaller scatter in strength properties may be expected, the value of γ_m in the ultimate limit state may however be put equal to 1.15.

[55] Latest wording BFS 1995:18.

Table a. Modification factor k_r for calculation of the resistance of structural timber and glued laminated timber in Service Classes 0, 1 and 2[1].

Strength classes	Action of shortest duration in a combination of actions[2]		
	P or A	B	C
f_m, f_t, f_c, E_R, G_R			
L40, K35	0.60	0.75	0.85
L30, K30, LK30	0.65	0.80	0.90
K24, LK20	0.70	0.85	1.00
K18, K12	0.75	0.90	1.00
$f_{t\,90}$			
all strength classes	0.40	0.60	0.80
$f_{c\,90}$, f_v			
all strength classes	0.60	0.75	0.85

(BFS 1998:39)

[1] For Service Class 3, the values are to be multiplied by a further factor of 0.85.

[2] The values refer to that action in a combination of actions which is of the shortest duration. P, A, B and C denote load duration classes in accordance with Subsection 5:22.

Table b. Modification factor k_r for calculation of the resistance of K plywood in Service Class 1[1].

Plywood strength class	Action of shortest duration in a combination of actions[2]		
	P or A	B	C
f_m, f_t, f_c			
P40	0.60	0.75	0.85
P30	0.65	0.80	0.90
P20	0.70	0.85	1.00
$f_{t\,90}$, $f_{c\,90}$, f_p, f_v			
all strength classes	0.60	0.75	0.85
E_R, G_R			
P40, P30	0.60	0.75	0.85
P20	0.70	0.85	1.00

[1] In Service Class 0 the values may be raised by 10% in relation to the values in Service Class 1. In Service Class 2 the values are to be multiplied by a factor of 0.7 and in Service Class 3 by a factor of 0.6. For tension parallel to the grain, the value of this factor may be put at 0.9 and 0.75 respectively.

[2] The values refer to that action in a combination of actions which is of the shortest duration. P, A, B and C denote load duration classes in accordance with Subsection 5:22.

Table c. Modification factor k_r for calculation of the resistance of K particleboard and flooring grade particleboard.

Action of shortest duration in a combination of actions [1]	Service Class 0	Service Class 1	Service Class 2 [2]
P or A	0.45	0.4	0.3
B	0.6	0.55	0.4
C	0.8	0.7	0.5

[1] The values refer to that action in the combination of actions which is of the shortest duration. P, A, B and C denote load duration classes in accordance with Subsection 5:22.

[2] The tabulated values apply for particleboard of Grade V313.

Table d. Modification factor k_r for calculation of the resistance of K-board

Action of shortest duration in a combination of actions [1]	Service Class 0	Service Class 1	Service Class 2 [2]
P or A	0.45	0.4	0.25
B	0.6	0.55	0.35
C	0.8	0.7	0.45

[1] The values refer to that action in the combination of actions which is of the shortest duration. P, A, B and C denote load duration classes in accordance with Subsection 5:22.

[2] For Strength Class K13 the tabulated values are to be multiplied by the factor 0.6.

Table e. Modification factor k_r for calculation of the loadcarrying capacity of a timber joint.

Joint Action of shortest duration in a combination of actions [1]	Service Class 0 & 1	Service Class 2	Service Class 3 [2]
Timber, plywood or steel in contact with timber			
P or A	0.7	0.7	0.6
B	0.8	0.8	0.7
C	1.0	1.0	0.8
Particleboard or fibreboard in contact with timber			
P or A	0.45	0.33	–
B	0.6	0.55	–
C	0.8	0.7	–

[1] The values refer to that action in a combination of actions which is of the shortest duration. P, A, B and C denote load duration classes in accordance with Subsection 5:22.

[2] For a nail or screw subject to a withdrawal force, the factor for Service Class 3 shall be multiplied by a further factor of 0.8.

:3122 *Tension*

General recommendation:

The resistance in tension parallel to the grain, R_{td}, may be calculated from formula (a) below. *(BFS 1998:39)*

$$R_{td} = f_{td} A \qquad \text{(a)}$$

The resistance in tension perpendicular to the grain, R_{t90d}, may be calculated from formula (b) and (c) below.

$$R_{t90d} = f_{t90d} A \qquad \text{(for structural timber and glued structural timber)} \qquad \text{(b)}$$

$$R_{t90d} = \left(\frac{V_0}{V}\right)^{0,2} f_{t90d} A \qquad \text{(for glued laminated timber)} \qquad \text{(c)}$$

(BFS 1998:39)

NOTATION

f_{td}	design value for tension parallel to the grain
f_{t90d}	design value for tension perpendicular to the grain
A	area of cross section
V_0	reference volume which may be taken as 0.01 m^3
V	uniformly stressed volume. *(BFS 1998:39)*

:3123[56] Compression

The resistance in compression shall be calculated with regard to the risk of stability failure and of crushing due to local pressure.

General recommendation:

For a homogeneous compression member of structural timber or glued laminated timber, the design resistance R_{cd} can be determined from formula (a) below.

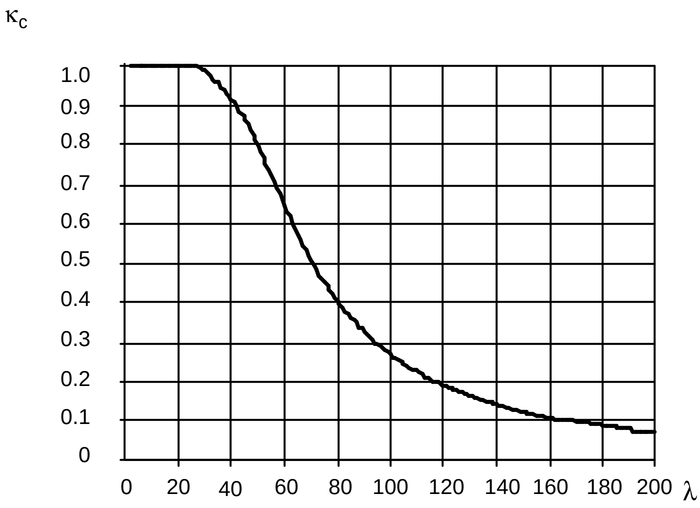
$$R_{cd} = \kappa_c f_{cd} A \qquad \text{(a)}$$

NOTATION

f_{cd}	design value of compressive strength parallel to the grain
A	cross sectional area of member
κ_c	reduction factor which takes account of the risk of buckling and can be determined from Figure (a) or from Formulae (b) – (e) below
λ	slenderness ratio of member = l_c/i, where l_c is the effective length of the member determined with respect to the degree of fixity and restraint in adjacent constructions, and i is the radius of gyration of the cross section.

[56] Latest wording BFS 1995:18.

Figure a. Reduction factor k_c for structural timber with E_{Rk}/f_{ck} = 300



$$k_c = \begin{cases} 1 & \text{for } l \le 27 \quad \text{(c)} \\ \dfrac{1}{k + \sqrt{k^2 - l_r^2}} & \text{for } l > 27 \quad \text{(d)} \end{cases}$$

$$k = 0.5\,(1 + b\,(l_{\mathrm{r}} - 0.5) + l_{\mathrm{r}}^2) \qquad \text{(d)}$$

$$\lambda_{\mathrm{r}} = \frac{\lambda}{\pi}\sqrt{\frac{f_{\mathrm{ck}}}{E_{\mathrm{Rk}}}} \qquad \text{(e)}$$

If the conditions in Sections 5:41 and 5:42 are satisfied, the value of β may be put equal to 0.2 for structural timber and to 0.1 for glued laminated timber.

The resistance in compression due to local pressure perpendicular to the grain may be calculated from formula (f) below.

$$R_{\mathrm{c90d}} = k_{\mathrm{c90}}\, f_{\mathrm{c90d}}\, A \qquad \text{(f)}$$

(BFS 1998:39)

NOTATION

F_{c90d} design value of compressive strength perpendicular to the grain

A cross sectional area of member

κ_{c90} magnifying factor which, for example, takes account of the loaded length, see e.g. SS-ENV 1995-1-1, 5.1.5. *(BFS 1998:39)*

:3124 *Bending*

General recommendation:

The design value of the resistance, R_{md}, in bending about a principal axis can be determined from formula (a) below.

$$R_{md} = \kappa_{inst} W f_{md} \quad \text{(a)}$$

NOTATION

f_{md} design value of bending strength

W section modulus according to the elastic theory

κ_{inst} reduction factor which takes account of the risk of lateral instability. If the conditions in Sections 5:41 and 5:42 are satisfied, κ_{inst} can be determined from Figure (a) or from Formulae (b) – (e) below.

Figure a. Reduction factor k_{inst}

$$k_{inst} = \begin{cases} 1 & \text{for } l_m \leq 0.75 \quad \text{(b)} \\ 1.56 - 0.75\, l_m & \text{for } 0.75 < l_m \leq 1.4 \quad \text{(c)} \\ \dfrac{1}{l_m^2} & \text{for } 1.4 < l_m \quad \text{(d)} \end{cases}$$

$$l_m = \sqrt{\frac{f_{md}}{s_{md\,cr}}} \qquad \text{(e)}$$

NOTATION

$?_m$ slenderness parameter in bending

f_{md} design value of bending strength

$s_{md\,cr}$ critical bending stress corresponding to lateral instability, calculated according to the elastic theory using the design values of the modulus of elasticity and shear modulus. *(BFS 1995:18)*

:3125 Shear

For members with sudden changes in cross section, for instance end notched beams, the effect of stress concentrations shall be taken into consideration.

:3126 Combined axial tension and bending

:3126 Combined axial tension and b

General recommendation:

Conditions (a) and (b) should be satisfied.

$$\frac{S_{mxd}}{R_{mxd}} + k_m \frac{S_{myd}}{R_{myd}} + \frac{S_{td}}{R_{td}} \leq 1 \qquad \text{(a)}$$

$$k_m \frac{S_{mxd}}{R_{mxd}} + \frac{S_{myd}}{R_{myd}} + \frac{S_{td}}{R_{td}} \leq 1 \qquad \text{(b)}$$

NOTATION

S_{mxd}, S_{myd}	design value of the effect of action due to bending moment about the x and y axes
S_{td}	design value of the effect of action due to tensile force
R_{mxd}, R_{myd}	design value of resistance in bending in accordance with Subsection 5:3124
R_{td}	design value of resistance in tension in accordance with Subsection 5:3122

For a rectangular cross section, k_m can be put equal to 0.7 and for other cross sections to 1.0.

:3127 Combined axial compression and b

General recommendation:

In combined bending and axial compression when there is no risk of buckling, i.e. if $\lambda \leq 27$, conditions (a) and (b) should be satisfied.

$$\frac{S_{mxd}}{R_{mxd}} + k_m \frac{S_{myd}}{R_{myd}} + \left(\frac{S_{cd}}{R_{cd}}\right)^2 \leq 1 \qquad \text{(a)}$$

$$k_m \frac{S_{mxd}}{R_{mxd}} + \frac{S_{myd}}{R_{myd}} + \left(\frac{S_{cd}}{R_{cd}}\right)^2 \leq 1 \qquad \text{(b)}$$

NOTATION

S_{mxd}, S_{myd}	design value of the effect of action due to bending moment about the x and y axes
S_{cd}	design value of the effect of action due to compressive force
R_{mxd}, R_{myd}	design value of resistance in bending in accordance with Subsection 5:3124
R_{cd}	design value of resistance in axial compression in accordance with Subsection 5:3123
k_m	reduction factor in accordance with Subsection 5:3126

In combined bending and axial compression when there may be a risk of buckling, i.e. $\lambda > 27$, conditions (c) and (d) should be satisfied.

$$\frac{S_{mxd}}{R_{mxd}} + k_m \frac{S_{myd}}{R_{myd}} + \frac{S_{cd}}{R_{cd}} \leq 1 \qquad \text{(c)}$$

$$k_m \frac{S_{mxd}}{R_{mxd}} + \frac{S_{myd}}{R_{myd}} + \frac{S_{cd}}{R_{cd}} \leq 1 \qquad \text{(d)}$$

NOTATION

$S_{mxd,}\, S_{myd}$	design value of the effect of action due to bending moment about the x and y axes, calculated without consideration of the deformation of the member
S_{cd}	design value of the effect of action due to axial compressive force
$R_{mxd,}\, R_{myd}$	design value of resistance in bending in accordance with Subsection 5:3124
R_{cd}	design value of resistance in compression in accordance with Subsection 5:3123
k_m	reduction factor in accordance with Subsection 5:3126

5:32 Design in the serviceability limit states

:321 Calculation of forces and moments

General recommendation:

The analytical model should be chosen as appropriate in accordance with Subsection 5:311.

If a combination of actions consists of actions of different duration according to Subsection 5:22, the total effect of the actions is to be calculated as the sum of the individual effects.

:322[57] Design values of material properties

The design values in the serviceability limit states shall be determined in accordance with general formula (a) below:

$$E_d = \frac{k_s\ E_k}{g_m} \qquad \text{(a)}$$

NOTATION

E_d design value of material property

κ_s modification factor which takes account of service class and the load duration class. Unless some other value is shown to be valid, the values of κ_s given in Tables (a) – (c) below for structural timber, glued laminated timber and structural boards shall be used

E_k characteristic value for calculations in the serviceability limit states, e.g. E_k (or G_k) for calculations of deformations in accordance with Subsection 5:23

γ_m partial factor for material properties which may be put equal to 1.0 in the serviceability limit states

General recommendation:

Unless it can be shown that other values are more appropriate, the values of κ_s should be taken from Tables (a)-(c) below for structural timber, glued laminated timber and wood based panels respectively. Joint slip may be calculated on the bases of the lowest modification factor of the jointed members.
(BFS 1998:39)

Table a. Modification factor k_s for calculation of the deformations of structural timber and glued laminated timber

Load duration class[1]	Service Classes 0 och 1	Service Class 2	Service Class 3
P	0.55	0.45	0.3
A	0.65	0.55	0.4
B	0.8	0.7	0.55
C	1.0	0.9	0.8

[1] P, A, B and C denote load duration classes as set out in Subsection 5:22.

Table b. Modification factor k_s for calculation of the deformations of structural plywood

Load duration class[1]	Service Classes 0 och 1	Service Class 2	Service Class 3
P	0.55	0.5	0.3
A	0.65	0.6	0.4
B	0.8	0.7	0.55
C	1.0	0.9	0.8

[57] The amendment means inter alia that the last sentence in the definition of κ_s is deleted.

[1] P, A, B and C denote load duration classes as set out in Subs ection 5:22.

Table c. Modification factor k_s for calculation of the deformations of K board, K-particleboard and flooring grade particleboard

Load duration class[1]	Service Classes 0 och 1	Service Cclass 2
P	0.3	0.2
A	0.4	0.3
B	0.55	0.4
C	0.8	0.55

[1] P, A, B and C denote load duration classes as set out in Subsection 5:22.

:323 Floor vibration

For a timber floor, account shall be taken of the risk of excessive vibration.

General recommendation:

The vibration susceptibility of a floor structure may be assessed in accordance with Boverket's handbook *Vibration, induced deformation and accidental actions*. For residential floors with solid timber beams in the direction of the main span, the following simplified calculation method may be used in assessing the vibration susceptibility of the floor.

The deflection of a single beam in a timber floor structure should not exceed 1.5 mm under the action of a point load of short duration (κ_s=1), the design value of which is 1.0 kN. The beam is assumed in the calculation to be simply supported and loaded at its midpoint. Any distribution of load to adjacent beams may be taken into account. *(BFS 1995:18)*

If interaction between the beams and floor boards is utilised in the calculation, the joints should be comprised in the additional inspection in accordance with Section 2:6. *(BFS 1995:18)*

5:33 Design by testing *(BFS 1995:18)*

The design value of the resistance R_d in the ultimate limit states shall be determined in accordance with formula (a) below.

$$R_d = \frac{k_r R_k}{g_{mp} g_n} \qquad (a)$$

NOTATION

R_k	characteristic value of resistance in accordance with Subsection 2:33 *(BFS 1995:18)*
κ_r	factor for reduction of the resistance with respect to the effect of moisture and load duration in accordance with Subsections 5:21 and 5:22
γ_n	partial factor for safety class in accordance with Subsection 2:115
γ_{mp}	partial factor for resistance which may be put equal to γ_m in accordance with Subsection 5:3121

General recommendation:

Design values in the serviceability limit states can be determined in accordance with Boverket's handbook *Design by testing*.

5:4 Materials

General recommendation:

General provisions regarding materials are given in Section 2:4.

The following subsections give examples of materials and fasteners which satisfy these requirements for timber structures.

5:41 Structural timber

General recommendation:

Where lateral instability can occur, the initial curvature of a structural element should be limited to 1/300 of the length.
(BFS 1998:39)

:411[58] Visually graded structural timber

General recommendation:

Visually graded sawn or planed structural timber is to be assigned to Strength Classes K30, K24, K18 or K12 if it is graded and marked as follows.

[58] The amendment means inter alia that the last sentence in the general recommendation is deleted.

K30: Marked T30 and graded in accordance with
— [1] *Instructions for grading and marking of T timber*
(stress graded timber), (Edition 5, 1981), published by the Swedish Stress Graded Timber Association,
— [2] *Grading rules for structural timber*, Annex A to Danish Standard DS 413 (Edition 4, 1982),
— [3] The Finnish grading rules drawn up by VTT, *Sahatavaran lujuuslajittelupas*,
— [4] *Quality requirements for structural timber*, Norwegian Standard NS 3080 (Edition 2, 1988).

Marked T3 and graded in accordance with
–
[5] SS 23 01 20

K24: Marked T24 and graded in accordance with
– [1],[2],[3] and [4] above, Marked T2 and graded in accordance with
[5] above.

K18: Marked T18 and graded in accordance with
– [1],[3] or [4] above,
Marked T1 and graded in accordance with [5] above.

K12: Marked T0 and graded in accordance with [5] above
Marked or unmarked timber
– which satisfies the requirements of SS 23 01 30,
– of Grade B or better in accordance with *Nordic Wood - Grading rules for sawn pine and spruce timber* 1994. The bow shall not, however, exceed 5 mm over a length of 2 m. *(BFS 1998:39)*

:412 Mechanically stress graded structural timber

General recommendation:

Mechanically stress graded structural timber which is graded and inspected in accordance with SS–EN 519 and marked in accordance with Chaper 4 of the approval rules (PFS 1978:3) of the Swedish Board of Physical Planning and Building, *Mechanically stress graded structural timber*, with the exception of d) in Clause 4:1, is regarded as structural timber in Strength Classes K35, K30, K24 and K18.

When applying the requirements for additional visual grading in SS–EN 519, Strength Class C18 in accordance with SS–EN 338 is to be substituted by Strength Class K18 according to Subsection 5:411. *(BFS 1998:39)*

:413 Finger jointed structural timber

General recommendation:

Finger jointed structural timber which is produced and inspected in accordance with SS–EN 385 and marked in accordance with Chapter 4 of the approval rules (PFS 1975:7) of the Swedish Board of Physical Planning and Building, *Finger jointed structural timber*, with the exception of the text regarding the Board's mark of approval in Clause 4:1, is regarded as structural timber in Strength Classes K35, K30, K24, K18 and K12. For structural timber in Strength Class K12, the strength of the joint should not be less than that of Strength Class K18. *(BFS 1998:39)*

:414 Round timber

General recommendation:

Round timber without loose rot or deep holes made by wood destroying insects is regarded as structural timber of Strength Class K30. Round timber for permanent structures should be free of bark.

:415 Glued structural timber *(BFS 1998:39)*

General recommendation:

Glued timber components made up of two or three laminates with the grain oriented along the longitudinal direction of the component are regarded as glued structural timber. Glued structural timber should be produced, inspected and marked in accordance with *Rules for Production Control of Glued Laminated Timber and Glued Structural Timber*, Swedish Council for Laminated Timber Control, 1997:1. *(BFS 1998:39)*

5:42 Glued laminated timber

5:42 Glued laminated timber

General recommendation:

Glued timber components made up of not less than four laminates with the grain oriented along the longitudinal direction of the component are regarded as glued laminated timber. Glued laminated timber should be produced, inspected and marked in accordance with *Rules for Production Control of Glued Laminated Timber and Glued Structural Timber*, Swedish Council for Laminated Timber Control, 1997:1.

Where lateral instability can occur, the initial curvature of a structural element should be limited to 1/500 of the length.
(BFS 1998:39)

5:43 Structural board

General recommendation:

Structural board of K plywood and K board should have the properties specified in, and should be produced, inspected and marked in accordance with, the approval rules (1975:5) of the Swedish Board of Physical Planning and Building, *Wood based board materials – manufacture and control*.

Structural board of K particleboard and flooring grade particleboard should have the properties specified in, and should be produced, inspected and marked in accordance with, Product Rules No 5, *Particleboard*, of the Nordic Committee for Building Regulations NKB.

5:44 Joints

:441 Mechanical connectors

General recommendation:

Wire nails with a cross section $d \leq 6$ mm and a least characteristic failure moment in accordance with Table (a) below should be used. For square and grooved nails d (mm) is the least side dimension and for round nails the diameter.

In bolted joints, bolts in Strength Class 4.6 in accordance with Swedish Standard SS 2265, and nuts in Strength Class 4 in accordance with SS 2268, should be used.

In bolted joints and in joints with screws, steel washers of not less than $0.3d$ thickness and a lateral dimension (diameter or length of side) not less than $3d$ should be used, where d is the bolt or screw diameter.

Screws complying with Swedish Standards SMS 1573-1575 and SS 2020 should be used.

Table a. Characteristic yield moments for wire nails

Type of nail	Yield moment (Nmm)
Square and grooved	$10\ (20-d)\ d^{3}$
Round	$6.7\ (20-d)\ d^{3}$

:442 Adhesives

General recommendation:

Adhesives which comply with the requirements for Adhesive Type I in accordance with Swedish Standard SS-EN 301 can be used for structures in all service classes. Adhesives which comply with the requirements for Adhesive Type II can be used for structures in Service Classes 0 – 2.

5:5 Execution and workmanship

General recommendation:

General provisions regarding execution and workmanship are set out in Section 2:5.

In this Section, examples are given of execution and workmanship which complies with these provisions.

5:51 Timber

General recommendation:

The face of a timber component which is joined to another timber should not have wane, loose knots or similar defects to such an extent that the strength of the joint is inadequate. Timber which splits, for instance while being nailed, is to be rejected.

Finger jointed structural timber may be used in a loadbearing structure provided that

a) the timber is produced, inspected and marked in accordance with the provisions of Subsection 5:413,

b) the structure is designed so that failure of a single finger joint does not cause collapse of essential parts of the structure as a whole.

The condition in b) is considered to have been complied with in structures comprising closely spaced beams or trusses, and implies that finger jointed structural timber should be used with care in structures in Safety Class 3.

Finger jointed structural timber should not be used in scaffolds or other structures subject to impact loading.

5:52 Joints

:521 Nailed joints

General recommendation:

In a joint between a structural (K) board and timber, the nails which transmit load should have adequate pointside penetration in the timber.

For nails of $d \geq 5$ mm, the timber should be predrilled using a drill of 0.8–0.9 d.

:522 Punched nail plate joints

General recommendation:

Punched nail plate joints may be made in accordance with the approval rules (1974:4), *Punched nail plate joints*, of the Swedish Board of Physical Planning and Building.

:523 Bolted and screwed joints

General recommendation:

Holes for bolts should be drilled so that the bolts must be forced in. Bolts should if necessary be retightened after the timber has dried.

Holes for screws should be drilled for the unthreaded portion so that there is a tight fit to the shank diameter, and for the threaded portion with a drill of 0.8 – 0.9 times the core diameter.

:524 Glued joints

General recommendation:

Glued joints in glued structures which are not to be classified as glued laminated timber in accordance with Subsection 5:42 should be constructed in accordance with the approval rules (PFS 1975:6), *Glued timber structures. Production and inspection*, of the Swedish Board of Physical Planning and Building.

In nail pressure gluing and screw pressure gluing, the appropriate adhesive and the distance between nails and screws should be determined on the basis of trials.

5:6 Supervision and control

The values of the partial factors γ_m and γ_{mp} set out in this section presuppose that the supervision and control specified in Section 2.6 is carried out. *(BFS 1995:18)*

5:61 Basic inspection

General recommendation:

Basic inspection of timber structures should comprise inspection of materials, joints, products and workmanship.

Site inspection should check that the quality and dimensions of joints, support details and timber conform to the requirements and conditions given on drawings and in other construction documents, and the execution and workmanship to the provisions of Section 5:5. In a joint the number of connectors should also be checked.

5:62 Additional inspection

General recommendation:

Additional inspection of timber structures should comprise checks on

- joints in glued laminated structures
- protection of timber in structures in contact with soil,
- glued joints used to transmit force, and
- erection of structures subject to production control which have been designed using the partial factor $\gamma_m = 1.15$.

6 MASONRY STRUCTURES

6:1 Requirements

General recommendation:

General requirements are set out in Section 2:1.

6:11 Durability

Masonry structures shall be designed, detailed and constructed with due regard to harmful degradation.

General recommendation:

Examples of harmful degradation are corrosion and frost attack.

Examples of materials which are considered to comply with the requirements concerning corrosion resistance are given for wall ties in Subsection 6:44 and for reinforcement in Subsection 6:45. Examples of exposure classes for reinforcement are given in Subsection 6:3128.

Bricks and blocks which are found frost resistant in tests by a method according to Table (a) below are considered to comply with the requirement concerning frost resistance. Clay bricks which, according to Section 5.6.1 of SS 22 01 11, are of Class F2 can be regarded as frost resistant. Clay bricks of Class F1, moderately frost resistant, may be used in facades in regions and in positions where the climatic conditions are moderate if

– earlier documented experiences with the type of clay bricks in question are good and

– the detailing of the structure has been designed or is otherwise judged to limit moisture influences.

Some examples of regions where the climatic conditions are severe are Driving Rain Zone 2, 3 and 4 according to *Moisture handbook – practice and theory*, Section 93:4. Driving rain is, however, only one of several factors which influence the durability of masonry. *(BFS 1998:39)*

Bricks and blocks shall be dimensionally stable.

General recommendation:

The requirement concerning dimensional stability in conjunction with changes in moisture content may be considered to have been complied with if the change in length in shrinkage tests by a method according to Table (a) below does not exceed the following values:

— mean value for test specimens 0.05%

— maximum individual value 0.06%.

Table a. Examples of suitable test methods for frost resistance and dimensional stability

Material	Frost resistance	Dimensional stability
Clay bricks and sand lime bricks	SIS 22 01 11	SIS 22 01 11
Concrete bricks, concrete blocks and lightweight aggregate blocks	SS 22 72 31	SS 22 72 31
Autoclaved aerated concrete blocks	–	SS-EN 680

(BFS 1998:39)

6:2 Design assumptions

General recommendation:

General design assumptions are given in Section 2:2.

6:21 Characteristic values of material properties for masonry structures

The characteristic values of the strength, modulus of elasticity and ultimate compressive strain of masonry structures which are set out in this section *(Section 6:2)* apply if the following conditions are satisfied:

— The cross sectional area of the masonry structure is not less than 0.04 m^2, allowance being made for chases and recesses.

— The masonry structure is laid to bond with the perpends staggered not less than 60 mm for bricks and not less than one quarter of the length of a block.

— The masonry structure is constructed with a least nominal thickness in accordance with Table (a) in Subsection 6:311.

— The masonry is constructed in accordance with the first clause of *Regulations for Execution and Workmanship,* of *Hus AMA 98* Chapter FS *(General Material and Workmanship Specifications for Buildings). (BFS 1998:39)*

General recommendation:

For masonry thinner than 150 mm a deduction from the cross section should also be made with respect to mortar joints if these are raked out or indented more than 3 mm.

:211[59] Compressive strength

The characteristic values set out in Table (a) below shall be applied for the compressive strength f_{ck} perpendicular to bed joints and parallel to any holes in bricks and blocks, and for the ultimate compressive strain ε_u under actions of long duration. These values apply if the following conditions are satisfied:

[59] Lastest wording BFS 1995:18. The amendment means inter alia that the value for compressive strength for hollow concrete block masonry with Class A mortar is deleted.

— The average joint thickness is not greater than 15 mm for bricks and blocks and not greater than 3 mm for thin joint masonry.

— The masonry is to be laid on a full bed of mortar except in the case of shell bedded clinker block masonry. For shell bedded clinker blocks where not more than the middle third of the joint is left unfilled, the tabulated values are to be multiplied by the factor 2/3. This compressive strength is in such a case assumed to be uniformly distributed over the entire width of the joint (the thickness of the wall). If not more than the middle sixth of the width of the bed joint is left unfilled in clinker block masonry, the compressive strength need not be reduced.

— The masonry is constructed with the perpends completely filled. Masonry of clinker blocks may however be constructed with the perpends completely unfilled, without any reduction in the vertical compressive strength ($f_{ck,tra}$). On the other hand, the compressive strength parallel to the bed joints is to be reduced when the perpends are unfilled. *(BFS 1998:39)*

General recommendation:

General recommendation:

Clinker blocks wider than 150 mm should be laid with shell bedding.

Table a. Characteristic values of the compressive strength f_{ck} and the ultimate compressive strain ε_u under actions of long duration for different masonry structures *(BFS 1995:18)*

Bricks/blocks	Strength Class	f_{ck} (MPa) Mortar Class according to SS 13 75 19				ε_u [1] (‰)
		A	B	C	D	
Clay bricks	15	5.8	4.6	3.3	1.3[4]	4
	25	7.5	6.0	4.3	1.8[4]	4
	35	8.9	7.1	5.0	2.3[4]	4
	45	10	8.0	5.7	2.3[4]	4
	55	11.1	8.9	6.3	2.3[4]	4
	65	12.1	9.7	6.9	2.3[4]	4
Sandlime bricks	25	–	6.0[3]	4.3[3]	–	4
Concrete bricks	25	7.5	6.0	–	–	4
Hollow concrete blocks	5	1.7	1.7	1.5	–	4
	10	2.4	2.4	2.0	–	4
Solid concrete blocks	10	3.8	3.8	3.0	–	4
	15	4.7	4.7	3.7	–	4
Autoclaved aerated concrete blocks [2]	1.7	–	1.1	1.1	–	4
	2.3	–	1.4	1.4	–	4
	3	–	1.7	1.7	–	4
	5	–	2.7	2.1	–	4
Clinker blocks	2	–	1.8	1.2	0.5[4]	2.5
	3	–	2.4	1.6	0.5[4]	2.5
	5	–	3.4	2.2	0.5[4]	2.5
	10	–	4.3	3.4	0.5[4]	2.5

(BFS 1998:39)

[1] The values apply for Class B or C mortar. For Class A mortar the values of ε_u shall be multiplied by the reduction factor 0.75 and for Class D mortar by the factor 2.1. *(BFS 1998:39)*

[2] The strength classes given correspond to Quality Groups 400, 450, 500 and 600. The designation for the quality group is based on the nominal dry density of the autoclaved aerated concrete in kg/m^3.

[3] Refers to masonry cement A as binder.

[4] On condition that the masonry is never subjected to temperatures below +0°C and its mean temperature over 24 hours is not under +5°C, neither during construction nor as finished structure, 40% higher values may be applied.See also Subsection 6:51. *(BFS 1998:39)*

If the average thickness of the bed joints is greater than 15 mm in brick or block masonry of strength class higher than 5, the tabulated values of f_{ck} shall be reduced. For 30 mm joints the tabulated values of f_{ck} shall be multiplied by the reduction factor 0.6. Linear interpolation is to be applied for joint thicknesses between 15 and 30 mm.

For blockwork of strength class not higher than 5, no reduction need be made for average joint thickness up to 20 mm. For 30 mm bed joints the tabulated values of f_{ck} shall be multiplied by the reduction factor 0.8. Linear interpolation is to be applied for joint thicknesses between 20 and 30 mm.

For loadbearing masonry acted upon by vertical loads, the joint thickness shall not exceed 30 mm on average or 35 mm for an individual joint.

For the characteristic strength *parallel* to bed joints in masonry of hollow bricks or hollow blocks, the tabulated values of f_{ck} shall be multiplied by the reduction factor 0.4. Also in the case of clinker block masonry laid on a full bed of mortar or shell bedded, with the perpends open, the tabulated values of f_{ck} shall be multiplied by the reduction factor 0.4 for compressive strength parallel to the bed joints.

:212 Flexural tensile strength

For masonry subject to flexure due to transverse action (wind action, earth pressure etc) the characteristic values of the flexural tensile strength, $f_{tk,par}$ parallel to the bed joints (vertical moment vector) and $f_{tk,tra}$ perpendicular to the bed joints (horizontal moment vector) tabulated below shall be applied unless other values can be shown to be applicable.

Table a. Characteristic values of the flexural tensile strength of masonry, $f_{tk,par}$ and $f_{tk,tra}$.

Bricks/blocks	Strength class	Mortar Class according to SS 13 75 19	$f_{tk,par}$ (MPa)	$f_{tk,tra}$ (MPa)
Hollow clay bricks	15—65	A—B	1.1	0.3
Solid clay bricks	15—65	A—B	1.1	0.25
Sandlime bricks	25	B	0.9	0.1
Concrete bricks	25	B	0.9	0.2
Hollow concrete blocks	5—10	A—B	0.4	0.2
Solid concrete blocks	10—15	A—B	0.4	0.2
Autoclaved aerated concrete blocks	1.7	B	0.1	0.1
	2.3	B	0.2	0.1
	3	B	0.25	0.2
	5	B	0.25	0.2
Clinker blocks	2	B	0.15	0.15
	3	B	0.3	0.15
	5	B	0.3	0.15
	10	B	0.3	0.15

(BFS 1995:18)

For masonry in Mortar Class C the tabulated values shall be multiplied by the reduction factor 0.8.

For blockwork masonry the tabulated values apply when the blocks are laid with the perpends staggered by half a block or by

250 mm. The tabulated values apply to masonry with filled joints. For clinker block masonry laid on shell bedding and with the perpends left open, the values shall be multiplied by the reduction factor 0.75, and for blocks laid on a full bed of mortar and with the perpends left open, only $f_{tk,par}$ shall be multiplied by the reduction factor 0.75.

For masonry wall beams subject to flexure in the plane of the wall due to the dead load of the wall and applied vertical load if any, the tabulated values of $f_{tk,par}$ shall be used as the characteristic value of the tensile strength in flexure in bonded masonry without soldier courses or brick on edge courses.

For wall beams with soldier courses or brick on edge courses the values of $f_{tk,tra}$ in Table (a) shall be used as characteristic values.

:213[60] Shear strength

General recommendation:

The characteristic value of shear strength parallel to bed joints,
$f_{vk,par}$ may be calculated from formulae (a)-(e) below:

For masonry consisting of

clay bricks $f_{vk,par} = 0.10 + 0.75\sigma_n$ but ≤ 1.0 MPa (a)

sandlime bricks $f_{vk,par} = 0.10 + 0.50\sigma_n$ but ≤ 0.6 MPa (b)

thin joint autoclaved aerated concrete blocks
$f_{vk,par} = 0.25 + 0.50\sigma_n$ but ≤ 0.8 MPa

clinker blocks $f_{vk,par} = 0.15 + 0.90\sigma_n$ but ≤ 1.1 MPa ((d)

concrete bricks, hollow and solid concrete blocks,
and autclaved aerated concrete blocks
$f_{vk,par} = 0.15 + 0.50\sigma_n$ but ≤ 0.6 MPa (e)

where σ_n is the mean compressive stress in the combination of actions concerned. *(BFS 1998:39)*

Formulae (a)– (e) apply for Mortar Class A or B. For Class C the value of $f_{vk,par}$ is to be multiplied by the reduction factor 0.8. For clinker blockwork laid on shell bedding and with open perpends, $f_{vk,par}$ is to be calculated from formula (b).

The characteristic value of shear strength, $f_{vk,tra}$, perpendicular to bed joints, i.e. in a vertical cross section, can be put equal to 0.8 MPa for Mortar Classes A, B and C. In applying this value it is assumed that the shear force is uniformly distributed over the cross sections of the bricks or blocks. The perpends should not therefore be included in the calculations.
(BFS 1998:39)

Formulae (a) – (e) presuppose a strength class not less than 15 for bricks and not less than 1.7 for blocks.

[60] Latest wording BFS 1995:18.

:214[61] Modulus of elasticity

The characteristic value E_k of the modulus of elasticity of masonry under short term loading shall be assumed constant for stresses between the design value of the flexural tensile strength, f_{td}, and 0.6 f_{cd}, where f_{cd} is the design value of the compressive strength.

General recommendation:

[61] Latest wording BFS 1995:18.

General recommendation:

For normal masonry in Mortar Class A, B or C, the approximate value is

$E_k = 400 f_{ck}$	for solid clay bricks and sandlime bricks	(a)
$E_k = 700 f_{ck}$	for hollow clay bricks and thin joint autoclaved aerated concrete blocks	(b)
$E_k = 1400 f_{ck}$	for clinker blocks	(c)
$E_k = 1000 f_{ck}$	for concrete bricks, hollow and solid concrete blocks, and autoclaved aerated concrete blocks	(d)

(BFS 1998:39)

Where a more accurate value of the modulus of elasticity is required, it should be determined by tests.

6:22 Deviations from size and shape

General recommendation:

Examples of deviations from size and shape which should be taken into consideration in design are given in Subsection 6:51. *(BFS 1998:39)*

6:3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions regarding design are set out in Section 2.3. *(BFS 1995:18)*

6:31 Design in the ultimate limit states

:311 Calculation of forces and moments

Masonry shall have the least nominal thickness set out in Table (a) below. The table presupposes that the deviations from size and shape are not greater than those specified in Subsection 6:51.

Table a. Least nominal wall thickness for masonry of different heights

Height of masonry	Least nominal wall thickness (mm)	
	Loadbearing wall	Veneer wall
Not more than 2 storeys, $\leq$ 6 m	85	60
More than 2 storeys, > 6 m	120	85

Cracking of the masonry shall be taken into consideration if they are of essential significance for the distribution of forces and moments in the structure. *(BFS 1998:39)*

Restraint forces shall be calculated with reference to the mode of action of the structure in the ultimate limit state.

:312 Calculation of resistance

:312 Calculation of resistance

:3121 Design values of material properties for masonry

The design values of the strength f_d and modulus of elasticity E_d of masonry in the ultimate limit states shall be determined from formulae (a) and (b) below.

$$f_d = \frac{f_k}{g_m \, g_n} \qquad \text{(a)}$$

$$E_d = \frac{E_k}{\gamma_m \, \gamma_n} \qquad \text{(b)}$$

NOTATION

f_k	characteristic value of the strength of the masonry in accordance with Subsections 6:211, 6:212 and 6:213
E_k	characteristic value of the modulus of elasticity of the masonry in accordance with Subsection 6:214
γ_m	partial factor for the resistance of the masonry in accordance with Subsection 6:3123
γ_n	partial factor for safety class in accordance with Subsection 2:115

If a high value of the modulus of elasticity is unfavourable in the ultimate limit states, E_d shall be put equal to E_k.

In determining the shear capacity R_v, consideration shall be given to the fact that, as set out in Subsection 6:213, the shear strength parallel to the bed joints is different from that perpendicular to these. The shear strength parallel to the bed joints, $f_{vd,par}$, may be utilised only in the part of the cross section where axial tensile stress does not simultaneously occur. *(BFS 1998:39)*

:3122 Design values of material properties for reinforc

In the ultimate limit states the design value of the tensile strength, f_{std}, shall be determined from formula (a) below.

$$f_{std} = \frac{f_{yk}}{g_m\, g_n} \tag{a}$$

NOTATION

f_{yk} characteristic value of strength for reinforcement. The value of f_{yk} for B500B and Ks 60 S shall be taken from Table (a) in Subsection 7:231 *(BFS 1998:39)*

γ_m partial factor for the resistance of reinforcement in accordance with Subsection 6:3123

γ_n partial factor for safety class in accordance with Subsection 2:115.

General recommendation:

For hot rolled reinforcement (e.g. B500B), the design value f_{scd} of the compressive strength should be assumed to have the same numerical value as the tensile strength f_{std}. For cold worked reinforcement the value of f_{scd} should be assumed to be equal to not more than 0.5 f_{std}. *(BFS 1998:39)*

The characteristic value of the modulus of elasticity E_{sk} of reinforcement shall be assumed to be 200 GPa.

In the ultimate limit states the design value E_{sd} shall be determined from formula (b) below.

$$E_{sd} = \frac{E_{sk}}{1.05\,\gamma_n} \qquad \text{(b)}$$

In design for accidental actions and with respect to progressive collapse, E_{sd} shall be put equal to E_{sk}.

:3123 Partial factor g_m for masonry, wall ties and reinforcement

The value of the partial factor γ_m for masonry, wall ties and reinforcement shall be determined from Table (a).

Table a. Prescribed partial factors γ_m for resistance in the ultimate limit states in general

	Partial factor γ_m			
	Execution Class I		Execution Class II	
	1	2	1	2
Masonry				
In general	2.3	1.8	2.9	2.3
Nonloadbearing walls subject to wind action	1.9	1.5	2.4	1.9
Wall ties				
Strength	1.9	1.5	2.4	1.9
Anchorage	2.5	2.0	3.1	2.5
Reinforcement				
Strength	1.9	1.5	–	–
Anchorage	2.5	2.0	–	–

The values of the partial factor γ_m in column 1 in each execution class apply for the design of masonry structures when the products used (bricks, blocks, mortar or reinforcement) have *not* been subject to factory production control in accordance with Section 1:4.

The values of the partial factor γ_m in column 2 in each execution class apply for the design of masonry structures when all the products used (bricks, blocks, mortar or reinforcement) have been subject to factory production control in accordance with Subsection 1:4.

:3124 Stress-strain

General recommendation:

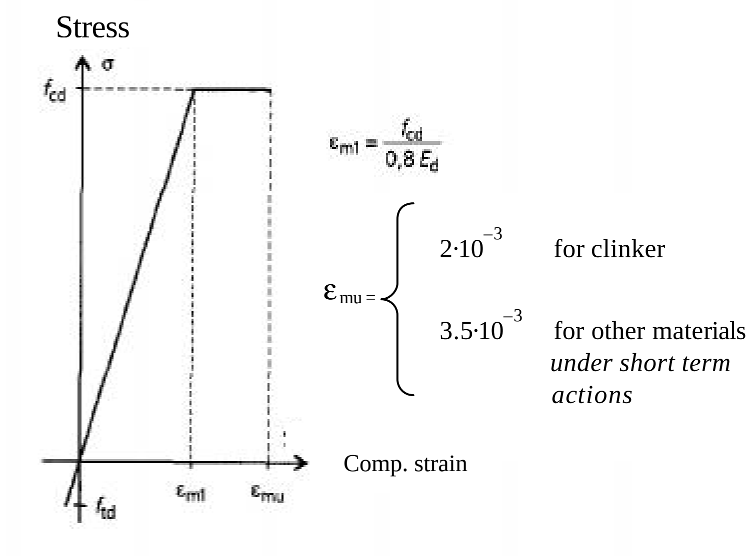
The stress-strain curve in Figure (a) below can be applied for masonry in calculating

- the moment capacity of reinforced masonry,
- deformations in reinforced masonry, and
- stability, but only for Mortar Classes A, B and C

and if the mean compressive strain is not greater than 0.6 f_{cd}.

(BFS 1998:39)

Figure a. Idealised stress-strain curve for masonry



In calculating the moment capacity of reinforced masonry, the compressive stress may be assumed constant and equal to f_{cd} within 0.8 times the depth of the compression zone, measured from the edge in compression. In more accurate calculations a stress-strain curve for masonry in accordance with that in Figure 2.4.5a in *BBK 94* may be used. An appropriate transition curve is then to be inserted so that it is tangential to the straight lines AB and CD.

For reinforcement, design may be based on a schematic stress-strain curve in accordance with Figure 2.5.5a-b in *BBK 94*. For cold worked reinforcement a strain greater than the prescribed limiting strain ε_g given in the relevant standard, reduced by the term 0.01, should not be assumed. *(BFS 1998:39)*

:3125 Masonry acted upon by vertical actions

General recommendation:

The design resistance R_{ncd} per unit length of the masonry should be calculated from formula (a) below.

$$R_{ncd} = b\,t\,f_{cd} \qquad \text{(a)}$$

NOTATION

b	reduction factor whose magnitude is determined by the eccentricity of the action and the slenderness of the masonry, see e.g. MUR 90, Section 4B:4.6. *(BFS 1995:18)*
t	thickness of masonry
f_{cd}	design value of the compressive strength in accordance with Subsections 6:211 and 6:3121

:3126 Masonry acted upon by transverse actions

General recommendation:

For a wall acted upon by transverse actions which is supported at least along three sides, the tensile strength of the masonry can be utilised. The design of such a wall can be based on plate action.

When a wall is supported along two vertical sides, i.e. in the case of wall strips spanning in one direction horizontally, the tensile strength in flexure $f_{td,par}$ parallel to the bed joints, in accordance with Subsections 6:212 and 6:3121, can be utilised.

For structures in Safety Classes 2 and 3, the resistance with respect to transverse actions shall not be based only on the tensile strength in flexure perpendicular to bed joints.

General recommendation:

For unreinforced masonry the resistance can be calculated in accordance with the elastic theory for an isotropic slab. In complicated cases, e.g. walls with openings, approximate solutions based on subdivision of the masonry into elements may be applied. Attention must be paid to the essential continuity and elasticity conditions.

The calculated resistance at the first flexural crack is normally to be taken as the ultimate limit. If after cracking the masonry can resist larger transverse actions, the collapse action is the ultimate limit.

As an alternative to calculation using the elastic theory, a method based on the mode of action of the masonry in the ultimate limit state may be applied.

The resistance with respect to horizontal actions may also be calculated on the basis of arching action without utilisation of the tensile strength. This applies on condition that the supports are sufficiently rigid so that the arching forces in the plane of the wall can be resisted.

:3127 Local pr

Local pressure shall be limited to ensure that splitting does not give rise to disruption of function or a reduction in the resistance or durability of the structure.

General recommendation:

Local pressure is to be considered to exist when the size of the contact area in the longitudinal direction of the masonry is less than twice the thickness of the masonry or less than one third of the length of the masonry.

:3128 Reinforced m

Masonry in a building of more than two storeys and masonry with in-situ reinforcement shall be constructed to Execution Class I. Masonry with in-situ reinforcement in single-dwelling houses of not more than two storeys and masonry reinforced only to take up movement forces may, however, be constructed to Execution Class II. Mortar in reinforced masonry shall be of Class A or B.
(BFS 1998:39)

General recommendation:

The moment capacity R_{md} of the masonry should be calculated from formula (a) below.

$$R_{md} = A_s\, d \left(1 - 0.5\,\frac{A_s}{bd}\cdot\frac{f_{std}}{f_{cd}}\right) f_{std} \qquad \text{(a)}$$

The moment capacity R_{md} calculated according to formula (a) presupposes that the cross section is normally reinforced and should therefore satisfy condition (b) below. *(BFS 1998:39)*

$$R_{md} \leq 0.3bd^2 \ f_{cd} \tag{b}$$

NOTATION

A_s area of tensile reinforcement
b width of wall strip or wall beam
d effective depth
f_{std},f_{cd} design values in accordance with Subsections 6:3122 and 6:3121

When formula (b) is applied to masonry of clinker blocks, the factor 0.3 should be replaced by 0.2. The mechanical reinforcement content of the cross section, *w*, may also be verified in the same way as for concrete structures at which e_{cu} is put equal to e_{mu} in accordance with Subsection 6:3124. *(BFS 1998:39)*

The shear capacity should be checked both parallel and perpendicular to the bed joints. The resistance of masonry in shear, R_{vd}, should be calculated from formula (c) below

$$R_{vd} = bd\, f_{vd} \tag{c}$$

where f_{vd} is the design value of shear strength in the direction concerned in accordance with Subsections 6:213 and 6:3121.

A masonry wall in which joint reinforcement is utilised to resist transverse action, e.g. wind action or earth pressure, may be designed as a reinforced wall strip spanning in one direction or as a two-way spanning slab.

Reinforcement in masonry shall be anchored so that it can in each section resist the forces which occur under design loading.

General recommendation:

For ribbed bars laid in mortar of Class A or B the bond stress may be assumed to be uniformly distributed over the surface area of the reinforcement. The characteristic value f_{bk} of bond strength should be assumed to be not greater than 1.0 MPa. *(BFS 1998:39)*

All reinforcement in the span should be carried not less than 200 mm beyond the edge of the support.

In masonry reinforced to resist concentrated loads greater anchorage lengths may be required.

Lap splices shall be constructed in such a way that the ends of bars have the required anchorage, and so that the force at the section can be transmitted from one bar to the other. The increased risk of splitting where laps are near one another shall be taken into consideration. Reinforcement in beams and walls required to carry load shall not be lapped.

General recommendation:

The lap length in reinforcement for transverse action should be not less than 500 mm. The midpoint of the lap should be not less than 1.0 m from the vertical support section.

In bed joints with two reinforcing bars the midpoints of the lap splices should be staggered by not less than 1.0 m. The same applies to adjacent lap splices at different levels in the masonry.

The limiting values of diameter, joint thickness and clear distance between reinforcing bars, set out in Table (a) below, shall apply to reinforcement in bed joints.

Table a. Limiting values of bar diameter, least joint thickness and least clear distance

Reinforcing steel	Bar diameter ∅ (mm)		Least joint thickness (mm)	Least clear distance (mm)[2]
	min	max		
Ribbed bar	6	8[1]	∅ + 7	2 ∅
Ladder and framework type reinforcement	3.5	6	∅ + 7	–

(BFS 1998:39)

[1] For reinforcement placed in special grooves in block masonry and surrounded by not less than 10 mm mortar, a bar diameter up to 12 mm may be used.

[2] Lapped bars are however to be placed in contact.

General recommendation:

The values of minimum cover set out in Table (b) below for different grades of reinforcement and for the exposure classes set out in Table (c) overleaf satisfy the requirements regarding corrosion resistance in Subsections 2:13 and 6:11.

Table b. Minimum cover in the horizontal direction for masonry reinforcement

Exposure Class	Cover (mm)			
	ob[1]	fz[2]	rf[3]	eb[4]
1. Environment of insignificant aggressivity to reinforcement	25	15	10	15
2. and 3. Environment of moderate/high aggressivity to reinforcement	–	–	15	–

[1] ob = untreated steel
[2] fz = zinc coated steel, min. Fe/Zn 45 SS 3192 *(BFS 1998:39)*
[3] rf = stainless acid resistant SS Steel 2340 or 2343, or cold drawn 18/8 steel grade SS 2331
[4] eb = steel treated with epoxy

Table c. Examples of exposure classes for masonry structures

Exposure Class	Type of structure
1 (A1)[1]	Internal walls,the inner leaves of cavity walls, the warm inside of block walls, basement walls with two stage waterproofing
2 och 3 (A2, A3)[1]	Other walls such as facades, basement walls and internal walls in aggressive industrial atmospheres

[1] Exposure classes according to *BBK 94* Subclause 7.3.2.2

:3129[62] Wall ties in veneer walls

Wall ties and their anchorage shall be designed for forces due to wind action and temperature movements in the masonry. *(BFS 1998:39)*

In design with respect to wind action any imposed deformations due to e.g. temperature movements in the masonry shall be taken into consideration.

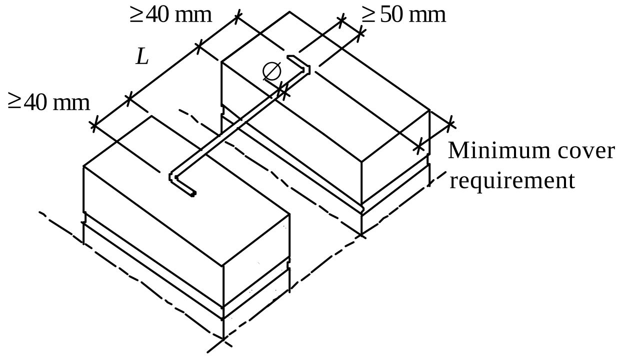
General recommendation:

The design values, which may be assumed for temperature movements in an arbitrary direction, are 0.25 mm/m for clay bricks and clinker blocks, 0.3 mm/m for sandlime bricks and 0.4 mm/m for concrete bricks. When these values are used in the same combination of actions as wind actions, the effect of fatigue may be assumed to have been taken into consideration. *(BFS 1998:39)*

Anchorage of wire type wall ties in completely filled joints should be effected by bending the end of the wall tie as shown in Figure (a) below. If the wall tie is of another type or it is fixed in some other way or if mortar of Class D is used, its anchorage and deformation properties should be determined by tests. *(BFS 1998:39)*

[62] The amendment means inter alia that the second paragraph in the general recommendation is deleted.

Figure a. Anchorage of wire type wall tie. *(BFS 1998:39)*



6:32 Design in the serviceability limit states

General recommendation:

The analytical model should as appropriate be selected in accordance with Sections 6:31 and 2:31. Forces and moments should however be determined by the elastic theory.

:321 Design values of material properties

General recommendation:

In calculating deformation due to actions of long duration, the design value of the fictitious modulus of elasticity of the masonry, $E_{d\varphi}$, should be determined from formula (a) below.

$$E_{d\varphi} = \frac{E_d}{1+\varphi} \tag{a}$$

where the creep coefficient φ can be put equal to 1.0 for brick masonry and to 2.0 in other cases.

In design in the serviceability limit states, the design value of the moduli of elasticity of the masonry and reinforcement may be put equal to the characteristic value.

General recommendation:

The values of other partial factors for design in the serviceability limit states are given in Subsection 2:322.

:322 Deformation and cracking

Masonry structures shall be designed and detailed with respect to the risk of cracking due to movements as a result of loading, initial shrinkage and moisture and temperature action in the masonry or adjoining elements of structure.

General recommendation:

Reinforcement provided to distribute cracking due to temperature and shrinkage movements should be lapped not less than 500 mm. The midpoint of the lap splice should be positioned not less than 1.0 m from a vertical support section. For adjacent lap splices at different levels in the masonry, the midpoints of laps should be staggered not less than 1.0 m.

The least cover of mortar in accordance with Subsection 6:3128 also applies to reinforcement provided to distribute cracking due to temperature and shrinkage movements.

6:33 Design by testing *(BFS 1995:18)*

The design value of resistance, R_d, for the ultimate limit states shall be calculated from the characteristic value R_k by formula (a) overleaf.

$$R_d = \frac{R_k}{\gamma_{mp}\,\gamma_n} \qquad \text{(a)}$$

NOTATION

R_k characteristic value of resistance

γ_{mp} partial factor for resistance in accordance with Subsection 6:3123

γ_n partial factor for safety class in acordance with Subsection 2:115

General recommendation:

Design values for the serviceability limit states can be determined in accordance with Boverket's handbook *Design by testing*.

6:4 Materials

General recommendation:

General provisions regarding materials are set out in Section 2:4.

The material properties of bricks/blocks and mortar shall be suited to one another in such a way that satisfactory bond and watertightness are achieved in the joints.

General recommendation:

Examples of combinations of bricks/blocks and mortar which satisfy the requirement that material properties should be suited to one another are given in Table (a) below.

Table a. Suitable combinations of bricks/blocks and mortar

Material	Element of structure	Mortar class in accordance with SS 13 75 19
Clay bricks	External walls	A[1], B, C[2], D[2]
	Internal walls	A[1], B, C[2], D[2]
	Reinforced vaults and wall beams	A, B
	Chimneys	B, C[2]
Sandlime bricks		B[3], C[3]
Concrete bricks		A, B
Concrete blocks	Foundation walls	A, B
	Other walls	A, B, C[2]
Autoclaved aerated concrete blocks	Foundation walls	B
	Other walls	B, C[2]
Clinker blocks	Foundation walls	B
	Other walls	B, C[2], D[2]

(BFS 1998:39)

[1] Normally, to be used only in reinforced masonry. *(BFS 1998:39)*
[2] According to Subsection 6:3128, shall not be used for reinforced masonry. Should not be used for work at temperatures below +5°.
For mortar of Class D see also Subsection 6:51. *(BFS 1998:39)*
[3] Refers to mortar with masonry cement A as binder.

6:41[63] Bricks and blocks

Bricks and blocks shall be assigned to strength classes in accordance with Subsection 6:21 if they are classified in accordance with Table (a) below. Formats and permissible dimensional deviations are set out in Table (b).

Bricks and blocks shall not contain harmful quantities of impurities or soluble salts which may affect the bricks, blocks or mortar.

General recommendation:

When the corrected sulphur content, S_ρ, is higher than 0.08% of lightweight aggregate used in lightweight aggregate blocks for masonry structures which may be subjected to high humidities, a sulphate resistant cement should be used. The sulphur content should be determined in accordance with SS 18 71 86.
(BFS 1998:39)

$$S_\rho = S\rho/1500 \qquad \text{(a)}$$

NOTATIONS

S sulphur content measured as the weight of total sulphur per weight of dry aggregate

ρ the dry bulk density of the lightweight aggregate in kg/m^3. *(BFS 1998:39)*

[63] Latest wording BFS 1995:18.

Table a. Strength classes for bricks and blocks

Bricks/blocks	Strength Class (MPa)	Marking	Strength required intests
Clay bricks	15[1], 25, 35, 45, 55[1], 65[1]	SIS 22 21 04	SIS 22 21 04
Sandlime bricks	25	SIS 22 21 05	SIS 22 21 05
Concrete bricks	25, 35, 45	SS 22 72 30	SS 22 72 30
Hollow concrete blocks	5, 10	SS 22 72 30	SS 22 72 30
Solid concrete blocks	10, 15	SS 22 72 30	SS 22 72 30
Autoclaved aerated concrete blocks	1.7, 2.3, 3 5	SS 22 81 50	SS 13 73 04
Clinker blocks	2, 3, 5, 10	SS 22 72 30	SS 22 72 30

[1] The compressive strength is to be determined in accordance with SIS 22 21 04. The mean value for 10 bricks shall be not less than 15, 55 and 65 MPa respectively, and the mean value for the five lowest of the ten values shall be not less than 12, 48 and 58 MPa respectively.

Table b. Formats and permissible dimensional deviations for bricks and blocks

Bricks/blocks	Format	Permissible dimensional deviations
Clay bricks	SIS 22 21 04	SIS 22 21 04
Sandlime bricks	SIS 22 21 05	SIS 22 21 05
Concrete bricks	SS 22 72 30	SS 22 72 30
Hollow and solid concrete blocks	SS 22 72 30	SS 22 72 30
Autoclaved aerated concrete blocks – for masonry – for thin joint masonry	 SS 22 81 50 SS 22 81 50	 SS 22 72 30 SS 22 81 50
Clinker blocks	SS 22 72 30	SS 22 72 30

6:42 Mortar

The constituent materials of mortar shall not contain harmful quantities of substances which have a deleterious effect on the mortar or materials with which the mortar comes into contact.

Mortar shall be assigned to Mortar Classes A, B or C if it is composed as stated in SS 13 75 19, Section 3 or *HusAMA 98* Table FS/1. They shall also fulfil the requirements on compressive strenght, setting, bleeding and air content given in SS 13 75 19, Sections 3, 4.3, 4.4 and 4.5.

Mortar shall be assigned to Mortar Class D if it is composed in accordance with what is stated for Mortar Class D in Clause 2.21 of the approval rules (PFS 1976:1) *Murbruk och murlim* (Masonry mortar and thin layer mortar) of the Swedish Board of Physical Planning and Building, or in Section 3 of SS 13 75 19. Mortar of Class D shall also comply with the requirements regarding compressive strength given in Section 2.28 of PFS 1976:1,tested in accordance with the method stated therein (Appendix 1, Section 1:18) and with the requirements in Sections 4.3, 4.4 and 4.5 of SS 13 75 19.

Binders for mortar shall comply with the requirements given in SS–ENV 413-1 (masonry cement), SS–ENV 459-1 (building lime) or ENV 197-1 (cement) subject to the additions and restrictions given in NAD(S)/ENV 197-1. *(BFS 1998:39)*

General recommendation:

Mortar is designated by the proportions by weight or volume of the mix.

Sand and water for mortar should have the propertes given in BBK 94 Section 7.2.4. The grading of the sand should, however, be as stated in Section 4.1 of SS 13 75 19. *(BFS 1998:39)*

Mortar for masonry structures in Execution Class I shall be proportioned by weight. Sand may however be measured by volume if its bulk density at the time has been determined.

6:43 Thin joint mortar

The term thin joint mortar refers to mortar for blocks laid with joints not more than 3 mm thick.

Binders for thin layer mortar shall comply with the requirements given in SS–ENV 413-1 (masonry cement), SS–ENV 459-1 (building lime) or ENV 197-1 (cement) subject to the additions and restrictions given in NAD(S)/ENV 197-1. *(BFS 1998:39)*

General recommendation:

Thin joint mortar should have the material properties and comply with the requirements regarding inspection and marking which are set out in Section 2.4, 3 and 4 of the approval rules (PFS 1976:1), *Murbruk och murlim* (Masonry mortar and thin layer mortar) , of the Swedish Board of Physical Planning and Building. *(BFS 1998:39)*

6:44 Wall ties

General recommendation:

Wall ties should be of a steel grade not lower than SS Steel 2340. For example, steel grades according to SS–EN 10 088 suitable for Exposure Classes M3/A3 and M4/A4 given in Table A.1 in NAD(S) to SS–ENV 1993-1-4 may be used. Examples of types of wall ties are given in SS 35 01 05. *(BFS 1998:39)*

6:45 Reinforcement

Reinforcement shall have such properties that, in interaction with the mortar, it can confer the intended performance and durability on the finished structure.

General recommendation:

Ladder type and framework type reinforcement, stainless ribbed bars or bars treated with epoxy may be used in masonry after special investigation. However, bars of Steel B500B and Ks 60 S may be used in Exposure Class 1, without special investigation. *(BFS 1998:39)*

6:5 Execution and workmanship

General recommendation:

General provisions regarding execution and workmanship are set out in Section 2:5.

Masonry structures shall be assigned to two execution classes, Class I and Class II. *(BFS 1995:18)*

Masonry in Execution Class I is masonry work which is directed and supervised by a person with special training in, and experience of, the construction of masonry structures. Masonry in Execution Class II is masonry work which is directed and supervised by a person with experience of the construction of masonry structures. *(BFS 1995:18)*

Masonry in a building of more than two storeys and masonry with in-situ reinforcement shall be constructed to Class I. In situ reinforced masonry in single-dwelling houses of not more than two storeys and masonry reinforced only for movement forces may however be constructed to Class II. *(BFS 1995:18)*

6:51 Masonry work

Bricks and blocks shall be laid on a full bed of mortar. Clinker blocks may however be laid on a full bed of mortar with unfilled perpends or be shell bedded with unfilled perpends. In shell bedding not more than the middle third of the joint is left free of mortar.

When mortar of Class D is used for masonry work it should be ensured that the masonry is protected from precipitation and temperatures below 0°C during the first 72 hours

– has a mean temperature over 24 hours which is higher than +5°C during the first 14 days, or alternatively has a mean temperature over 24 hours which is higher than +10°C during the first 6 days. *(BFS 1998:39)*

General recommendation:

Adjacent to floor slabs, the deviation of masonry from vertical alignment should be not more than 12 mm in Class I and not more than 20 mm in Class II. See BBK 94 where the vertical alignment, for example, corresponds to *a* in Figure 8.9.5.a and *e* in Figure 8.9.5c. *(BFS 1998:39)*

The highest value of the straightness deviation of masonry e_0, see Figure 8.9.5a of BBK 94, should be not more than h/300 for masonry in Class I and not more than h/200 in Class II. *BFS 1998:39)*

6:6 Supervision and control

The values of the partial factors γ_m and γ_{mp} set out in this section presuppose that the supervision and control specified in Section 2.6 is carried out. *(BFS 1995:18)*

6:61 Acceptance inspection on delivery

General recommendation:

On delivery, materials should be identified with respect to their strength class or quality group, origin and the inspection and control they have undergone. *(BFS 1995:18)*

:611[64] Basic inspection

General recommendation:

Basic inspection of bricks and blocks for products which have not been subjected to factory production control in accordance with Section 1:4 should also comprise tests in accordance with Table (a) below regarding density, dimensions and compressive strength and where recommended dimensional stability and frost resistance.

Basic inspection of mortar for products which have not been subjected to factory production control should also comprise tests in accordance with Table (a) below of the particle size distribution and humus content of the sand. If air or some other workability enhancement admixture is added at the mortar factory or on the site, the air content of the mortar mix should also be tested. Alternatively, the compressive strength of the mortar may be determined on test specimens made on the site. Tests should be carried out before masonry work begins and continuously to the extent required.

Where this is considered necessary, tests should also be made to determine the properties of the binder and water, and the clay and mud content and petrographic composition of the sand.

[64] Latest wording BFS 1995:18.

Table a. Suitable scope of tests on materials for masonry structures in different execution classes

Masonry	Bricks/blocks	Mortar (binder, sand and water)
Execution Class I	To be tested	Mortar and sand to be tested. The binder and water are not normally tested.
Execution Class II	To be tested	Sand is to be tested for buildings of more than two storeys. Other materials are not normally tested.

One sample series may represent not more than the number of bricks and blocks of the same type of consignment as indicated in Table (b) below. However, at least one sample series should be taken from each type of bricks or blocks of a consignment. If it can be shown that bricks or blocks, which have been delivered to a construction site at several occasions, belong to the same lot, then one sample series may represent all deliveries made to the same construction site of that particular lot, although with the limitations given in Table (b) below. A lot, in this respect, refer to bricks or blocks of the same type which have been manufactured at the same occasion and under identical condition.

Suitable methods for tests on the binder are given in SS–ENV 413-1, SS–ENV 459-1 and ENV 197-1 subject to the additions and restrictions given in NAD(S)/ENV 197-1. Examples of test methods for bricks and blocks are given in Table (c) below.

Delivery control of sand should be made in accordance with Subclause 9.2.2.3 of BBK 94. Suitable methods for tests of the humus content and the siltration content of the sand are given in SS 13 21 20 and SS 13 21 21 respectively. The particle size distribution (grading curve) of the sand should be determined with the method given in SS 13 21 23. For tests of the waterquality, in accordance with Subclause 7.2.5 of BBK 94, the methods given in SS 13 41 11 and SS 13 75 20 may be used.

A suitable method for the determination of the compressive strength of mortar of Classes A, B and C is given in SS 13 75 20. For mortar of Class D see Subsection 6:42. The air content of the mortar mix can be determined in accordance with SS 13 41 11.
(BFS 1998:39)

Table b. Suitable scope of tests during basic inspection of bricks and blocks not subject to factory production control

Bricks/ blocks	Max. number of bricks/ blocks in consignment for which a sample series shall be taken	Number of bricks/blocks per sample series in tests for			
		Density and size	Comp. strength	Dimensional stability	Frost resistens
Clay and sandlime bricks	300 000	10	10	–	10, 24[1], 30[1]
Concrete bricks	35 000	8	8	3	3
Solid and hollow concrete blocks	35 000	8[2]	8[2]	3[3]	3
Autoclaved aerated concrete blocks and clinker blocks	35 000	8[2]	8[2]	3[3]	–

(BFS 1998:39)

1 For sandlime bricks, 10 bricks are to be tested for frost resistance. For clay bricks with a nominal height of 87 mm, 24 bricks are to be tested for frost resistance, and for clay bricks with a nominal height of 62 mm, 30 bricks are to be tested.

2 13 blocks for consignments of more than 35 000 bricks/blocks.

3 Tests are not required if the blocks have been stored protected from the wet for not less than 21 days prior to use. *(BFS 1998:39)*

Table c. Examples of suitable test methods for bricks and blocks

Product	Method description
Clay bricks	SIS 22 01 11
Sandlime bricks	SIS 22 01 11
Concrete bricks	SS 22 72 31
Hollow concrete blocks	SS 22 72 31
Solid concrete blocks	SS 22 72 31
Autoclaved aerated concrete blocks	SS 13 73 06, SS 13 73 08-09, SS-EN 678, NS-EN 679, SS-EN 680
Clinker blocks	SS 22 72 31

6:62 Supervision of execution and workmanship

General recommendation:
Supervision of work with masonry in different execution classes should be carried out in accordance with Section 6:5.
(BFS 1998:39)

:621 Basic inspection

General recommendation:
Execution and workmanship should be subjected to particular checks with regard to the filling and thickness of mortar joints, the dimensional deviations of the masonry, and the anchorage lengths and cover of the reinforcement. The placing and number of wall ties in veneer walls should be checked against the drawings or other construction documents.

:622 Additional inspection

General recommendation:
Additional inspection of masonry structures should comprise checks on

- dimensional deviations and the length of bearings in loadbearing walls in buildings of more than two storeys,
- anchorage of leaves in veneer walls of a height greater than 12 m above ground level,
- other properties, constructional details etc which the designer considers are particularly significant for the safety, function and durability of the masonry structure.

7 CONCRETE STRUCTURES

The provisions of this section apply to loadbearing structures of ordinary concrete or lightweight aggregate concrete. The provisions apply to both plain and reinforced in-situ or precast structures with both reinforcing or prestressing steel. The provisions do not however apply to structures of autoclaved aerated concrete, no-fines concrete or other special types of concrete.

General recommendation:

The term ordinary concrete refers to concrete with cement and mineral admixtures as the binder and with rock material as the aggregate.

Structures of autoclaved aerated concrete which are constructed and inspected in accordance with the Approval Rules (PFS 1980:3), *Lightweight Concrete Products*, of the Swedish Board of Physical Planning and Building comply with the requirements for loadbearing structures in Section 2.

7:1 Requirements

General recommendation:

General requirements are set out in Section 2:1.

7:11 Durability

Concrete structures shall be designed, detailed and constructed in such a way that harmful degradation is prevented. This shall be done by assigning the elements of structure to the appropriate exposure class and by taking the required action to ensure that the structure shall resist the expected attacks.

General recommendation:

The risk of frost attack on the concrete should be taken into consideration in accordance with Subclause 7.3.2.1, and the risk of reinforcement corrosion in accordance with Subclause 7.3.2.2, of *BBK 94*.

7:12 Watertightness

Concrete structures which are expected to be subjected to water pressure on one side only shall have an adequate degree of watertightness.

General recommendation:

The degree of watertightness is dependent on constructional details and on the composition, casting and curing of the concrete. The requirement concerning the composition of concrete may be considered to have been complied with if the provisions of Subclause 7.3.4 of *BBK 94* are taken into consideration.

7:2 Design assumptions

General recommendation:

General design assumptions are set out in Section 2:2.

7:21 Actions

General recommendation:

Appropriate values of ψ_1 for determination of actions of long duration in accordance with Subsection 2:21 are given in Subclause 2.2.2 of *BBK 94*.

7:22 Characteristic values of material properties for concrete

With respect to its compressive and tensile strength, concrete shall be assigned to different strength classes.

General recommendation:

The standardised strength classes with respect to compressive strength are K16, K20, K25, K30, K35, K40, K45, K50, K55, K60, K70 and K80; for lightweight aggregate concrete there are the additional classes K8 and K12.

The numerical value of the strength class designation denotes the specified compressive strength f_K in MPa, determined by compressive tests on 150 mm cubes in accordance with Swedish Standard SS 13 72 10, and evaluated in accordance with Subclause 7.3.3.2 of *BBK 94*. *(BFS 1995:18)*

The standardised strength classes with respect to tensile strength are T1.0, T1.5, T2.0, T2.5, T3.0, T3.5 and T4.0.

The numerical value of the strength class designation denotes the specified tensile strength f_T in MPa, determined by splitting 150 mm cubes or cylinders of 150 mm diameter at the standard age, and with the tensile strength put equal to 0.8 times the splitting strength.

:221 Compressive strength

The characteristic values of the compressive strength of concrete, f_{cck}, set out in Table (a) below shall be used in calculating the design material properties.

In order that characteristic values > 21.5 MPa may be utilised, Execution Class I is required, and for values > 11.5 MPa, Execution Class I or II. In order that characteristic values > 56.5 MPa may be utilised, a special investigation is required.

Table a. Characteristic values[1] of the compressive strength f_{cck} of concrete

Strength class[2]	f_{cck} (MPa)	Strength class[2]	f_{cck} (MPa)
K 8	5.5[3]	K 40	28.5
K 12	8.5[3]	K 45	32.0
K 16	11.5	K 50	35.5
K 20	14.5	K 55	39.0
K 25	18.0	K 60	42.5
K 30	21.5	K 70	49.5
K 35	25.0	K 80	56.5

(BFS 1995:18)

[1] The characteristic values in the table are considered to correspond to 85% of the lower 5% fractile of the compressive strength of concrete cylinders of 150 mm diameter and 300 mm height, stored in water at 20 ± 2°C up to the time of testing and tested in accordance with ISO 4012. The correction factor 0.85 takes account of long term effects.

[2] When strength classes other than the standardised ones are used, $f_{cck} = 0.7 f_K$ where f_K is the required compressive strength of 150 mm cubes.

[3] Apply only for lightweight aggregate concrete. *(BFS 1995:18)*

:222[65] Tensile strength

If the strength of the concrete is checked only by compressive tests in accordance with Subsection 7:22, the characteristic values f_{ctk} of the tensile strength of concrete set out in Table (a) below shall apply.

In order that characteristic values > 1.05 MPa may be utilised, Execution Class I or II is required, and for values > 1.60 MPa, Execution Class I is required.

In order that characteristic values > 2.65 MPa may be used, a special investigation is required.

Table a. Characteristic values of the tensile strength f_{ctk} of concrete

Strength class	f_{ctk} (MPa)	Strength class	f_{ctk} (MPa)
K 8	0.75[1]	K 40	1.95
K 12	0.90[1]	K 45	2.10
K 16	1.05	K 50	2.25
K 20	1.20	K 55	2.40
K 25	1.40	K 60	2.50
K 30	1.60	K 70	2.60
K 35	1.80	K 80	2.65

[1] These values for lightweight aggregate concrete are to be factored as set out below.

[65] Latest wording BFS 1995:18.

For lightweight aggregate concrete the tabulated values shall be reduced by multiplication by the factor

$$0.3 + 0.7 \frac{r}{2\,400} \tag{a}$$

NOTATION

ρ density of the lightweight aggregate concrete (kg/m^3) determined from specimens cured and tested at the standard age in accordance with SS 13 72 10. *(BFS 1998:39)*

If the tensile strength of the concrete is checked by splitting tests in accordance with Subsection 7:22, the characteristic value f_{ctk} may in the serviceability limit states be put equal to the numerical value of the designation of the tensile strength class concerned. The specified tensile strength of the concrete shall in such a case be designated by one of the classes T1.0 – T4.0 according to Subsection 7:22. Tensile strength classes shall not however be applied in Execution Class III. In order that characteristic values > 1.5 MPa may be used, Execution Class I is required.

:223[66] Modulus of elasticity

When structures of ordinary concrete are loaded moderately rapidly, the characteristic values of the modulus of elasticity of concrete set out in Table (a) below shall be applied unless other values are shown to be applicable. The tabulated values refer to concrete without an air entraining admixture.

In order that characteristic values > 38.5 MPa may be used, a special investigation is required.

General recommendation:

In conjunction with rapid processes, e.g. oscillations, the values should be multiplied by 1.2.

Table a. Characteristic values of the modulus of elasticity E_{ck} of concrete

Strength class	E_{ck} (GPa)	Strength class	E_{ck} (GPa)
K 8	23.0[1]	K 40	32.0
K 12	24.5[1]	K 45	33.0
K 16	25.5	K 50	34.0
K 20	27.0	K 55	35.0
K 25	28.5	K 60	36.0
K 30	30.0	K 70	37.5
K 35	31.0	K 80	38.5

[1] These values for lightweight aggregate concrete are to be factored as set out below.

[66] Latest wording BFS 1995:18

For lightweight aggregate concrete the value of E_{ck} shall be reduced by multiplication by $\rho/2400$ where ρ is the density of the lightweight aggregate concrete (kg/m^3) determined from specimens cured and tested at the standard age in accordance with SS 13 72 10. *(BFS 1998:39)*

General recommendation:

> The characteristic value E_{ck} of the modulus of elasticity of concrete may be assumed constant for stresses between f_{ctd} and 0.6 f_{ccd}.

7:23 Characteristic values of material properties for reinforcing and prestressing steel

:231[67] Tensile strength

The characteristic value of the tensile strength of reinforcing and prestressing steel shall be equal to the lower 5% fractile of the upper yield stress or the 0.2% proof stress for the material.

Characteristic values of f_{yk} for standardised reinforcement are set out in Table (a) below.

Table a. Characteristic strength values f_{yk} for standardised reinforcement

Reinforcement type/ Reinforcement designation	Material requirements in accordance with SS	V: Hot rolled K: Cold worked	Limiting strain ε_g	Dim. and shape requirements in accordance with SS	Range of dimensions (mm)	f_{yk}[1] (MPa)
Plain bars Ss 26 S	14 14 11	V		21 25 11	6 – 32	270
Ribbed bars B500B[1]	SS-ENV 10080	V	0.05[2]	SS-ENV 10080	6 – 40	500
Ks 60 S	14 21 68	V		21 25 15	6 – 16 (16) – 25	620 590
Mesh Ns 50	14 13 86	K	0.03	21 18 45 21 25 18	4 – 11	510
Nps 50	14 13 87	K	0.03	21 18 45 21 25 19	5 – 12	510
B500B[1]	SS-ENV 10080	V	0.05[2]	SS-ENV 10080	6 – 16	500

(BFS 1998:39)

1 Rules for acceptance inspection are included in the standard.

2 ε_g is denoted A_{gt} in SS-ENV 10 080. *(BFS 1998:39)*

67 Latest wording BFS 1995:18. The amendment means inter alia that the reinforcement types Ks 40, Ks 40 S, K 500 and Ks 60 are deleted.

:232 Modulus of elasticity

Unless some other value is shown to be applicable the characteristic value E_{sk} of the modulus of elasticity for reinforcement shall be assumed to be 200 GPa. The characteristic value for prestressing tendons shall be determined on the basis of test results relating to the steel grade in question.

7:24 Prestressing forces

The characteristic value of the effective prestressing force shall on every occation be considered to be the nominal value.
(BFS 1998:39)

General recommendation:

The term effective prestressing force refers to the force in the tendon for a notional loading situation where the stress in the concrete at the level of the tendon is zero. *(BFS 1998:39)*

7:25 Deviations in size and shape

Tolerances relating to the dimensions of the cross section and the position of the reinforcement shall be taken into consideration in accordance with a) or b) below.

a) If the selected tolerances do not exceed the normal values and if the main dimension of the cross section is not less than 150 mm, deviations from nominal dimensions need not be taken into consideration in design. In conjunction with stability failure the same applies if the main dimension of the cross section in the direction of deflection is not less than 250 mm. *(BFS 1995:18)*

General recommendation:

Normal values of execution and workmanship tolerances are set out in *BBK 94* Clause 8.9. *(BFS 1995:18)*

b) If the conditions in a) above are not satisfied, deviations from the nominal dimensions shall be given special attention in design. The design values of strength and stiffness may in such a case be increased by multiplication by the factor 1.1 for concrete and 1.05 for reinforcement.

In designing columns and similar elements of structure in compression, the assumed values of load and support eccentricities, initial curvature, initial inclination, initial skewness etc shall be determined in view of the specified tolerances.

General recommendation:

If the normal values of tolerances are selected in accordance with *BBK 9*4 Subclause 8.9.5, the design assumptions set out in *BBK 9*4 Subclause 3.4.2.3 should be applied.

Tolerances smaller than normal values should not be applied.

7:3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions regarding design are set out in Section 2.3. *(BFS 1995:18)*

7:31 Design in the ultimate limit states

The provisions of this subsection *(Subsection 7:31)* relate to beams, columns, frames, arches, slabs, walls, shear panels, foundations and similar elements of structure of normal type and of usual cross section and details.

:311 Calculation of forces and moments

The assumed distribution of forces and moments in a structure shall satisfy equilibrium conditions and shall be such that the structure is capable of adjusting to the assumed distribution when undergoing deformation.

General recommendation:

With due regard to the requirements of the above mandatory provision, design may be based on the elastic theory or the limit states theory.

Methods for the selection of analytical models are given in *BBK 94* Subclause 3.2.1.

The following conditions apply for application of the limit states theory:

a) The resistance of the structure shall not be governed by brittle failure at the design value of the load, inclusive of the effect of deformation action.
b) Parts of a structure in which forces or moments are assumed to reach their limiting values prior to final failure shall have such deformation capacity that the stipulated redistribution of forces and moments can take place.
c) The resistance of the structure shall not be governed by stability failure at a load less than the design value, inclusive of the effect of deformation action.
d) The risk of incremental plastic rupture shall be taken into consideration.

General recommendation:

Examples of the way in which these requirements can be satisfied are given in *BBK 94* Subclause 3.2.3.

Cracking of the structure shall be taken into consideration if it is significant.

The effect due to alternative unfavourable load positions shall be taken into consideration.

General recommendation:

For floor constructions in residential buildings and other buildings subject to comparable conditions, the effect of unfavourable load position need be taken into consideration only in conjunction with the curtailment of reinforcement over supports.

:312 Calculation of resistance

With regard to the requirement regarding ductility in the ultimate limit states, concrete structures shall be designed so that the tensile forces which occur (e.g. due to bending moments) are resisted by reinforcement. Exceptions from this requirement may however be made in the following cases:

a) An element of structure in Safety Class I may be constructed without reinforcement.
b) An element of structure may be constructed without reinforcement if shrinkage and temperature variations may be expected to be small and if a tensile failure is not expected to have very serious consequences.
c) For a structure which satisfies the requirements in the ultimate limit states even after tensile failure (cracking), no reinforcement is required for the tensile forces concerned.
d) For special tensile forces in conjunction with shear, torsion, anchorage and local pressure, and in joints.

General recommendation:

For case d) appropriate design methods are given in Sections 3 and 6 of *BBK 94*.

:3121 Design values of material pro

The design values of material properties in the ultimate limit states shall be determined from formulae (a) - (c) below.

$$f_d = \frac{f_k}{h\, g_m\, g_n} \qquad \text{(a)}$$

$$E_d = \frac{E_k}{h\, g_m\, g_n} \qquad \text{(b)}$$

NOTATION

f_k characteristic value of strength in accordance with Subsections 7:22 and 7:23

E_k characteristic value of modulus of elasticity in accordance with Subsections 7:223 and 7:232 *(BFS 1995:18)*

η factor which takes account of systematic differences between material properties obtained in tests and the properties of the real structure. For concrete, $\eta = 1.2$ and for reinforcement, $\eta = 1.0$.

γ_m partial factor for resistance

γ_n partial factor for safety class in accordance with Subsection 2:115.

In the ultimate limit states the value of the product $\eta\gamma_m$ for concrete shall be put equal to 1.5 in determining strength values and to 1.2 in determining the modulus of elasticity.

For reinforcement and prestressing tendons the corresponding value of the product $\eta\gamma_m$ shall be put equal to 1.15 in determining strength values and to 1.05 in determining the modulus of elasticity.

For precast concrete units in Execution Class I which have been subjected to factory production control, strength values 5% higher may however be permitted if a separate additional inspection of the position of reinforcement is carried out.

In design with respect to accidental action, progressive collapse and fire, the value of the product $\eta\gamma_m$ for concrete may be put equal to 1.2 in determining strength values and to 1.0 in determining the modulus of elasticity.

For reinforcement and prestressing tendons the corresponding value of the product $\eta\gamma_m$ may be put equal to 1.0 in determining both strength values and the modulus of elasticity.

If a combination of actions comprises an action of clearly short term nature, the value of f_{ccd} may in design with respect to accidental actions and progressive collapse be multiplied by the factor 1.1.

General recommendation:

The term action of clearly short term nature refers here to an action which occurs only a few times and which attains values near the characteristic value for not more than 1 minute. These are usually actions of impact character, and the increase in the value of f_{ccd} is therefore mainly applicable in conjunction with certain accidental actions. The provisions regarding short term action may also be applied to piles subjected to forces from pile hammers. *(BFS 1995:18)*

If *a high value of the tensile strength of concrete* is unfavourable, the following shall be used as design value:

$$f_{cth} = 1.5\, f_{ctk} \tag{c}$$

(BFS 1995:18)

General recommendation:

The value of f_{ctk} may be taken from Table (a) in Subsection 7:222 even if f_{ctk} is determined by testing in other contexts.

If a *high value of the modulus of elasticity of concrete* is unfavourable in the ultimate limit states, the value of E_{cd} should be put equal to E_{ck}.

The design value of the shear modulus G_{cd} of concrete may be assumed to be equal to $0.4E_{cd}$.

Poisson's ratio for concrete may be assumed to be equal to 0.2. In most cases Poisson's ratio may however be ignored, i.e. it may be assumed to be equal to zero.

Design values of the compressive strength of reinforcement and prestressing tendons can be determined in accordance with Subclause 2.5.2 of *BBK 94*.

:3122 F

Structures subject to fatigue actions shall be designed and detailed in view of the risk of fatigue failure. Stresses shall be calculated in the same way as in the serviceability limit states.

General recommendation:

Examples of design methods with respect to fatigue are given in Clause 3.3 of *BBK 94*. Strength values for concrete in conjunction with fatigue may be determined in accordance with Subclause 2.4.3 of *BBK 94*. Strength values for reinforcement in conjunction with fatigue may be determined in accordance with Subclause 2.5.3 of *BBK 94*.

:3123 Stress-strain

General recommendation:

The stress-strain curve for concrete may be selected in accordance with Subclause 2.4.5 of *BBK 94* or based on test results. The stress-strain curve for hot rolled and cold worked reinforcing steel and prestressing tendons may be taken to be that in Subclause 2.5.5 of *BBK 94*. For cold worked reinforcing steel and prestressing tendons, design in the ultimate limit states and design with respect to progressive collapse may be based on a stress-strain curve in accordance with Subclause 2.5.5 in *BBK 94*.

:3124 Creep and shrinkage of co

In determining the creep of concrete, account shall be taken of the relative humidity of the environment, the composition and treatment of the concrete, the dimensions of the element of structure, the age of the concrete when action is applied, and the time from application of action to the time under consideration.

General recommendation:

The creep of concrete may be determined in accordance with Subclause 2.4.7 of *BBK 94*.

In determining the shrinkage of concrete, account shall be taken of the relative humidity of the environment, the composition and treatment of the concrete, the dimensions of the element of structure, and the age of the concrete measured from the time of casting. The effect of uneven shrinkage shall be taken into consideration.

General recommendation:

The shrinkage of concrete may be determined in accordance with Subclause 2.4.6 of *BBK 94*.

:3125[68] Prestressing forces and relaxation

The design value of the effective prestressing force shall be obtained from its characteristic value by multiplication by a partial factor which shall be put equal to 1.0 unless special conditions warrant some other value. *(BFS 1998:39)*

Calculation of prestressing data shall be based on the real relationship between stress and strain.

General recommendation:

Methods for calculation of effects of friction and time dependent effects are given in Subclauses 2.7.2 and 2.7.3 of *BBK 94*. *(BFS 1998:39)*

For prestressing tendons the relaxation of steel shall be taken into consideration.

General recommendation:

Relaxation should be taken into consideration in accordance with Subclause 2.5.6 of *BBK 94*.

:3126 Bending moment with or without longitudinal force

The limitations which apply for the deformation capacity of the concrete, the reinforcement and prestressing tendons shall be taken into consideration.

General recommendation:

Distribution of strain should be based on Subclause 3.6.2 of *BBK 94*.

In calculations for cracked concrete, the concrete shall normally be assumed not to resist tensile forces.

General recommendation:

The distribution of compressive stress may be based on Subclause 3.6.4 in *BBK 94*. Design of uncracked concrete should be based on Subclause 3.6.3 of *BBK 94*.

[68] Latest wording BFS 1995:18.

The longitudinal force on a cross section shall be assumed to have a least eccentricity which shall be put equal to 1/30 of the transverse dimension in the principal direction concerned. Eccentricity shall be measured from the centroid of the uncracked section without regard to reinforcement and need not occur simultaneously in two principal directions.

General recommendation:

An eccentricity less than 20 mm should not however be assumed.

:3127 Shear force and torsional m

The risk of brittle failure shall be taken into consideration.

General recommendation:

Examples of design methods for shear force are given in Clause 3.7 of *BBK 94*, and examples of design methods for torsional moment in Clause 3.8.

:3128 Anchorage and arrangement of reinforc

In order to prevent brittle failure, reinforcement shall be anchored and arranged so that there is a probability of at least 95% that the upper yield stress or 0.2% proof stress can be attained before anchorage failure or bond failure occurs.

General recommendation:

Examples of design methods for the anchorage of reinforcement are given in Subclause 3.9.1 of *BBK 94*.

Reinforcement shall at every section be capable of resisting the force which occurs at design load, due consideration being given to the effect of inclined cracks.

General recommendation:

Examples of methods for the curtailment of reinforcement are given in Subclause 3.9.2 of *BBK 94*.

Lap splices shall be made so that the ends of the bars have the required anchorage in the concrete, and so that force can be transmitted from one bar to the other. The increased risk of splitting and spalling in conjunction with laps near one another shall be borne in mind.

General recommendation:

Examples of methods for the lapping of reinforcement are given in Subclause 3.9.3 of *BBK 94*.

The radii of bends shall be sufficiently large with respect to the bending characteristics of the reinforcement and the risk of splitting of the concrete.

General recommendation:

Examples of methods for determining bending radii are given in Subclause 3.9.4 of *BBK 94*.

The concrete cover shall have adequate thickness with respect to the anchorage and lapping of the reinforcement and shall provide the required protection against corrosion and in certain cases fire.

General recommendation:

The thickness of cover should be selected in accordance with Subclause 3.9.5 of *BBK 94*.

The distance between parallel reinforcing units shall be sufficiently large with respect to the anchorage and lapping of the reinforcement and with respect to the placing and compaction of the concrete.

General recommendation:

Suitable distances between parallel reinforcing units are given in Subclause 3.9.6 of *BBK 94*.

The requirements for bundled reinforcement are the same as for single bars. Bundles shall be arranged so that all constituent bars can be surrounded by concrete and any gaps filled by concrete. Special attention shall be given to the risk of splitting near bends and where several bars are placed in the same plane.

General recommendation:

Examples of methods for bundling reinforcement are given in Subclause 3.9.7 of *BBK 94*.

:3129 Local pressure and transmission of force through joints

Local pressure shall be limited so that

— indentation does not occur to such an extent that the function of the structure is affected by the local deformation, and

— splitting does not occur to such an extent that splitting affects the function of the structure or reduces its resistance or durability. *(BFS 1995:18)*

General recommendation:

Examples of design methods are given in Clause 3.10 of *BBK 94*.

In calculating forces and moments in joints, it shall be borne in mind that the strength and the deformation properties may be different for joints and precast units, for different joints, and for different parts of the same joint.

Design shall be based on the mode of action of the joint type in transmitting force.

General recommendation:

Examples of design methods are given in Clause 3.11 of *BBK 94*.

7:32[69] Design in the serviceability limit states

The provisions of this subsection (Subsection 7:32) refer to beams, columns, frames, arches, slabs, walls, shear panels, foundations and similar elements of structure of normal type and of usual cross section and details.

General recommendation:

Design in the serviceability limit states refers primarily to deformations and cracking but can in special cases refer also to other conditions, for instance wear.

Analytical models should as appropriate be selected in accordance with Subsection 7:311. Forces and moments should however be determined by the elastic theory.

In the serviceability limit states the design values of material properties may be determined from formulae (a) - (c) below:

$$f_d = f_k \quad \text{(a)}$$

$$E_d = E_k \quad \text{(b)}$$

NOTATION

f_k characteristic value of strength in accordance with Subsections 7:22 and 7:23

E_k characteristic value of the modulus of elasticity in accordance with Subsections 7:223 and 7:232.

However, if a *high value of the tensile strength of concrete* is unfavourable, the following shall be taken as the design value:

$$f_{cth} = 1.5 f_{ctk} \quad \text{(c)}$$

General recommendation:

The value of f_{ctk} may in this case be taken from Table (a) in Subsection 7:222 even if the value of f_{ctk} in other contexts is determined by testing.

If under actions of long duration a high compressive stress occurs in the concrete, a special investigation of the magnitudes of creep deformations and their influence is required.

General recommendation:

If the compressive stress is limited in accordance with Subclauses 4.4.1 and 4.4.2 of *BBK 94*, such an investigation is not required.

During the stressing operation, the stress in prestressing tendons shall be limited to values such that the force can be reliably checked by measurement of extension, and in such a way that there is no risk of fracture of the prestressing steel. *(BFS 1998:39)*

[69] Latest wording BFS 1995:18.

General recommendation:

Appropriate limiting values during the stressing operation are given in Subclause 4.4.3 of *BBK 94*.

7:33 Design by testing *(BFS 1995:18)*

In the ultimate limit states the design value R_d shall be determined from formula (a) below.

$$R_d = \frac{R_k}{\gamma_{mp}\,\gamma_n} \qquad \text{(a)}$$

NOTATION

R_k characteristic value of resistance in accordance with Subsection 2:33

γ_{mp} partial factor for resistance according to Table (a) below

γ_n partial factor for safety class in accordance with Section 2:1. *(BFS 1995:18)*

Table a. Values of partial factor γ_{mp} for concrete structures

Critical property	Plain concrete	Reinforced concrete
Tensile strength of concrete	2.0	1.5
Compressive strength of concrete	1.5	1.5
Tensile strength of reinforcement	–	1.15
Instability failure	2.0	1.7

General recommendation:

In the serviceability limit states, design values can be determined in accordance with Boverket's handbook *Structural design by testing*.

7:34 Documentation

General recommendation:

Guidance regarding drawings and other design documents for concrete structures is given in Clause 1.4 of *BBK 94*.

7:4 Materials

General recommendation:

General provisions regarding materials are given in Subsection 2:4.

The materials for concrete, the concrete mix, the hardened concrete and the reinforcement and prestressing tendons, shall have such properties that the finished structure has the intended resistance, stability and durability. *(BFS 1995:18)*

General recommendation:

These properties should be verified by testing or in some other appropriate manner.

7:41 Materials for concrete

The materials for concrete shall not contain harmful quantities of constituents which may have an adverse effect on the properties or function of the concrete or reinforcement and/or prestressing steel.

In case of doubt regarding the suitability of a constituent material, it shall be shown by special investigation that the structure has satisfactory resistance, stability and durability and the other intended properties.

General recommendation:

Constituent materials should comply with the material properties set out in Clause 7.2 of *BBK 94*.

7:42 Concrete mix

The composition of the concrete mix shall be such that the concrete can completely fill the formwork and surround the reinforcement when poured and shall also remain homogeneous during the time it is handled.

7:43 Reinforcement, prestressing tendons and embedded fasteners

Reinforcement and prestressing tendons shall have such properties that, in interaction with concrete, they can confer a ductile mode of action on the structure at failure.

General recommendation:

In order that a ductile mode of action may be possible at failure, the characteristic value of the limiting strain of reinforcement and prestressing tendons should be not less than 3.0%, and the characteristic value of the ultimate tensile strength to yield stress ratio should be not less than 1.08.

In structures where the effect of displacement of supports or other induced action is negligible, reinforcement and prestressing tendons with a limiting strain of not less than 2.5% may however be used.

Reinforcement with structural and positional welds and welded mesh reinforcement shall in general be fabricated so that the regions affected by welding have an ultimate strength so much above the yield stress of the reinforcement that ductile failure can occur. *(BFS 1998:39)*

Mechanical reinforcement splices, end anchorages and the anchorages of embedded fasteners shall have an ultimate strength so much above the yield stress of reinforcement that ductile failure can occur.

General recommendation:

The methods for evaluation of tests given in Subsections 7.5.2 - 7.5.4 of *BBK 94* should be applied.

7:5 Execution and workmanship

General recommendation:

General provisions regarding execution and workmanship are set out in Section 2:5. Examples of suitable execution and workmanship are given in Section 8 of *BBK 94*.

During construction, a diary shall be kept which documents the work done, precipitation, temperature and other observations of significance for the quality of the finished structure.

7:51 Manufacture of the concrete mix

Concrete shall be proportioned and manufactured so that it has a homogeneous, uniform quality and a consistency which is adapted to the working method in question. The temperature of the concrete mix shall be limited so that no harmful effects occur.

Manufacture of the concrete mix shall be assigned to Classes I, II and III on the basis of the requirements concerning qualifications, supervision, equipment and transport, as well as uniformity and accuracy in manufacture. The most stringent requirements apply for Class I. *(BFS 1995:18)*

General recommendation:

A concrete mix manufactured in accordance with Subclauses 8.4.1.2 - 8.4.1.4 of *BBK 94* complies with the requirements for Manufacture Classes I-III.

In making concrete on the building site, the guidelines in Subclause 8.4.4 of *BBK 94* should be observed.

7:52 Work with concrete

The concrete mix shall be transported, placed, compacted and cured so that it remains homogeneous, without harmful cracking, and so that the finished structure has the intended resistance, stability and durability.

Construction joints shall be designed and made so that the finished structure has the required strength, durability and watertightness.

General recommendation:

Curing should be carried out in accordance with Subclause 8.5.2.4 of *BBK 94*.

Concrete work shall be assigned to Execution Classes I, II or III in view of the requirements concerning qualifications, supervision and uniformity and accuracy in workmanship. The most stringent requirements apply for Execution Class I. *(BFS 1995:18)*

Only concrete mix in Manufacture Class I shall be used for Execution Class I.

Only concrete mix in Manufacture Class I or II shall be used in Execution Class II.

General recommendation:

Concrete work carried out in accordance with Subclauses 8.5.1.2 - 8.5.1.4 of *BBK 94* complies with the requirements for Execution Classes I-III.

7:53 Construction and removal of formwork

Formwork shall be constructed so that the finished structure has the intended shape and function.

Formwork shall not be removed before the concrete has attained the necessary stiffness and strength and there is no risk of harmful cracking.

General recommendation:

Removal of formwork should be carried out in accordance with Clause 8.2 of *BBK 94*.

7:54 Reinforcement and prestressing tendons

Bending radii shall be sufficiently large to ensure that there is no risk that concrete will be crushed or split and to avoid cracking and other damage to the reinforcement.

General recommendation:

Bending of reinforcement should be carried out in accordance with Subclause 8.3.1 of *BBK 94*.

Welding of structural reinforcement shall be carried out so that welded joints and reinforcing bars have the necessary strength and ductility, due consideration being given to the special risks associated with different welding methods.

Welding of reinforcement in structures subject to fatigue actions shall be carried out so that the fatigue strength is not jeopardised.

General recommendation:

Welding of reinforcement should be carried out in accordance with Subclause 8.3.2 of *BBK 94*.

When reinforcement, prestressing tendons and cable ducts are placed, care shall be taken to ensure that these are not damaged and are free from harmful substances, and so clean that the intended bond can be attained.

Reinforcement, prestressing tendons and cable ducts shall be placed and fixed in position in such a way that after concreting they are in their intended positions in accordance with the drawings and within the specified tolerances.

General recommendation:

Positional reinforcement should be placed so that it has the requisite concrete cover as protection against corrosion and so that other reinforcement or tendons are in their intended positions.

Prestressing tendons should be tensioned in accordance with Subclause 8.3.4 of *BBK 94*.

7:55 Precast concrete units

If precast units must be lifted at special lifting points, these shall be marked.

It shall be possible for the bearing length to be checked after erection, which may necessitate special marking.

General recommendation:

Manufacture, marking, storage, handling and erection of precast concrete units should be carried out in accordance with Clauses 8.6 and 8.7 of *BBK 94*.

7:56 Special concreting procedures

General recommendation:

Guidance for underwater concreting, grouting, shotcreting and vacuum treatment is given in Clause 8.8 of *BBK 94*.

7:6 Supervision and control

The values of the partial factors γ_m and γ_{mp} set out in this section presuppose that the supervision and control specified in Section 2:6 is carried out. *(BFS 1995:18)*

The supervision and control specified in this section relate to work in Execution Classes I and II and in Manufacture Classes I and II.

7:61 General *(BFS 1995:18)*

General recommendation:

The necessary trial mixes should be made and the constituent materials, the concrete mix, the hardened concrete, and the reinforcement and/or prestressing tendons, should be continually checked. Control should be carried out in accordance with Section 9 of *BBK 94*. *(BFS 1995:18)*

The results of strength tests should be evaluated in accordance with Subclause 7.3.3.2 of *BBK 94* for continual checks and in accordance with Subclause 7.3.3.3 for strength tests on the finished structure.

:611 Basic inspection

General recommendation:

Examples of appropriate action in conjunction with basic inspection are given in Subclause 9.6.3 of *BBK 94*.

:612 Additional inspection

General recommendation:

Examples of appropriate action in conjunction with additional inspection are given in Subclause 9.6.4 of *BBK 94*.

7 :62 Acceptance inspection of concrete on delivery

General recommendation:

Acceptance inspection of concrete which has undergone factory production control should be carried out in accordance with Subclause 9.3.3 of *BBK 94*.

Acceptance inspection of concrete which has not undergone factory production control should be carried out in accordance with Subclause 9.3.4.3 of *BBK 94*. *(BFS 1995:18)*

7 :63 Acceptance inspection of reinforcement and prestressing tendons on delivery

General recommendation:

During acceptance inspection the reinforcement and prestressing tendons should be identified with regard to type, material grade, origin and control and, if this is particularly specified, charge. *(BFS 1995:18)*

Acceptance inspection of reinforcement and prestressing tendons which had undergone factory production control should be carried out in accordance with Subclauses 9.4.3 and 9.4.5.1 of *BBK 94*.

Acceptance inspection of reinforcement and prestressing tendons which had not undergone factory production control should be carried out in accordance with Subclauses 9.4.4 and 9.4.5.2 of *BBK 94*.

7:64 Acceptance inspection of precast concrete units

General recommendation:

Acceptance inspection of precast concrete units

which had undergone factory production control should be carried out in accordance with Subclause 9.5.3 of *BBK 94* and that of units which had not undergone factory production control in accordance with Subclause 9.5.4.

7:65 Supervision of execution and workmanship

General recommendation:

Supervision of work with concrete in different execution classes should be carried out in accordance with Subclause 9.6.2 of *BBK 94*. Control of concrete made on the building site should be carried out in accordance with Subclause 9.2.3 of *BBK 94*.

[73] Latest wording BFS 1995:18. The amendment means inter alia that SS-steel 2614, 2615, 2624, 2625, 2632, 2634, 2642, 2644, 2652, 2654, 2662 and 2664 in Table a are deleted and steel in classes with the designation Fe, with note, in Table b, also are deleted.

8 STEEL STRUCTURES

The provisions of this section relate to loadbearing structures of steel (carbon steels, carbon manganese steels, microalloyed steels, quenched and tempered steels, thermomechanically rolled steels, cold forming steels and stainless structural steels).

General recommendation:

Structures of thin gauge cold formed steel sheet, designed, constructed and inspected in accordance with *StBK-N5, Swedish Code for Thin Gauge Steel Structures 79*, comply with the requirements for loadbearing structures set out in Section 2.

8:1 Requirements

General recommendation:

General requirements are set out in Section 2:1.

8:11 Ductility

Steel structures shall be designed, detailed and constructed so that they have ductility properties such that a rapid increase in stress or a local stress concentration does not cause failure of the structure.

General recommendation:

The requirement concerning ductility may be considered to have been complied with if the structure is made of materials of properties in accordance with Subclauses 7:21 and 7:22 of *Swedish Regulations for Steel Structures BSK 94*.

8:12 Durability

Steel structures shall be designed, detailed and constructed with due regard to the risk of corrosion, wear and similar phenomena.

General recommendation:

Provisions regarding corrosion protection are set out in Subsection 8:56.

8:2 Design assumptions

General recommendation:

General design assumptions are set out in Section 2:2.

8:21 Actions

A decision is to be made in each individual case whether actions other than those referred to in Subsection 2:21 are to be regarded as fatigue actions.

An action which gives rise to fewer than 10^3 stress cycles during the service life of the structure need not be regarded as a fatigue action.

8:22 Characteristic values

The basic values of strength and other properties which are set out in this subsection presuppose materials which comply with the requirements for materials in Section 8:4.

:221[73] Values of strength

For general structural steel, the characteristic values, f_{yk} for the upper yield strength or 0.2% proof strength, and f_{uk} for the ultimate tensile strength, shall be taken from tables (a) - (e) below. *(BFS 1998:39)*

General recommendation:

Corresponding designations according to SS-EN 10 002-1 are R_{eH}, $R_{p0.2}$ and R_m respectively. SS-ISO 3898 uses the designation $f_{y,sup}$ for the upper yield stress.

The required minimum value f_{yk} is approximately equal to the 1% fractile.

Table a. Characteristic strength values of SS steels

SS-steel Ductility Class (Quality Class)[1]			Material thickness	Characteristic strength	
B	D	E	(mm)	f_{uk} (MPa)	f_{yk} (MPa)
1312[2]			— 16	360	240
1412[2]			— 16	430	270
2172[2]	2174[2]		— 16	470	320
			(16) — 40	470	310
			(40) — 100	470	300
2132[2]	2134[2]		— 16	470	360
			(16) — 35	470	350
			(35) — 50	470	340
			(50) — 70	470	330
2142[2]	2144[2]		— 16	490	390
			(16) — 35	490	380
			(35) — 50	490	370
			(50) — 70	490	360

(BFS 1998:39)

[1] According to International Institute of Welding, Document No. 367-71. *(BFS 1998:39)*
[2] Refers only to circular hollow sections, VKR and KKR.

Table b. Characteristic strength values of steels in accordance with SS-EN 10 025+A1

SS–EN 10 025+A1 Ductility Class (Quality Class)[1]		Material thickness	Characteristic strength	
B	D	(mm)	f_{uk} (MPa)	f_{yk} (MPa)
S235JRG2	S235J2G3	— 16	340	235
		(16) — 40	340	225
		(40) — 100	340	215
S275JR	S275J2G3	— 16	410	275
		(16) — 40	410	265
		(40) — 63	410	255
		(63) — 80	410	245
		(80) — 100	410	235
S355J0[4]	S355J2G3[2, 4]	— 16	490	355
		(16) — 40	490	345
		(40) — 63	490	335
		(63) — 80	490	325
		(80) — 100	490	315

(BFS 1998:39)

[1] According to International Institute of Welding, Document No 367-71.

[2] Delivery condition N according to SS-EN 10 025 for long products.

[4] See general recommendations regarding carbon equivalent in Subsection 8:225.

Table c. Characteristic strength values of steels in accordance with SS-EN 10 113

SS-EN 10 113 Ductility Class (Quality Class)[1]		Material thickness	Characteristic strength	
D	E	(mm)	f_{uk} (MPa)	f_{yk} (MPa)
S355N	S355NL	— 16	470	355
		(16)— 40	470	345
		(40)— 63	470	335
		(63)— 80	470	325
		(80)— 100	470	315
S355M	S355ML	— 16	450	355
		(16)— 40	450	345
		(40)— 63	450	335
S420M	S420ML[2]	— 16	500	420
		(16)— 40	500	400
		(40)— 63	500	390
S460M	S460ML[2]	— 16	530	460
		(16)— 40	530	440
		(40)— 63	530	430

(BFS 1998:39)

[1] According to International Institute of Welding, Document No 367-71.

[2] A special investigation should be made for welded structures subject to tensile stress at service temperatures below -20°C.

Table d. **Characteristic strength values of steel in accordance with SS-EN 10 137** *(BFS 1998:39)*

SS-EN 10 137 Ductility Class (Quality Class)[1]		Material thickness (mm)	Characteristic strength	
D	E		f_{uk} (MPa)	f_{yk} (MPa)
S460QL	460QL1	– 50	550	460
		(50) – 100	550	440
S500QL	S500QL1	– 50	590	500
		(50) – 100	590	480
S550QL	S550QL1	– 50	640	550
		(50) – 100	640	530
S620QL	S620QL1	– 50	700	620
		(50) – 100	700	580
S690QL	S690QL1	– 50	770	690
		(50) – 100	760	650

BFS 1998:39)

[1] According to International Institute of Welding, Document No 367-71. *(BFS 1998:39)*

Table e. **Characteristic strength values of steel in accordance with SS-EN 10 149.** *(BFS 1998:39)*

SS-EN 10 149 Ductility Class (Quality Class)[1]	Material thickness (mm)	Characteristic strength	
D		f_{uk} (MPa)	f_{yk} (MPa)
S355MC	1,5 – 20	430	355
S420MC	1,5 – 20	480	420
S500MC	1,5 – 16	550	500

(BFS 1998:39)

[1] According to International Institute of Welding, Document No 367-71. *(BFS 1998:39)*

The classification into ductility classes (quality classes) according to Tables (b), (c), (d) and (e) may be regarded as a general recommendation.

For steel grades of other types, the value of f_{yk} shall be determined on the basis of tests relating to the upper yield strength or 0.2% proof strength for test specimens taken parallel to the rolled direction. *(BFS 1998:39)*

General recommendation:

Characteristic strength values for other steel grades of thin gauge types are given in SS–ENV 1993-1-3. *(BFS 1998:39)*

:222 Values of strength in conjunction with fatigue action

The mechanical properties in conjunction with fatigue action shall be determined with regard to the magnitude and number of stress variations, the effect of notches, and workmanship.

The characteristic fatigue strength selected shall be such that it is not greater than the mean value less twice the standard deviation obtained in fatigue tests on test specimens of the same detailing and notch action.

:223 Modulus of elasticity, shear modulus and Poisson's ratio

Unless other values are shown to be more correct, the characteristic values E_k for the modulus of elasticity and G_k for the shear modulus shall be put equal to 210 GPa and 81 GPa respectively.

General recommendation:

For stainless steel it can normally be assumed that E_k = 190 GPa and G_k = 73 GPa.

In structures which presuppose interaction between steel and concrete, the modulus of elasticity of the reinforcement may be given the same characteristic value as that applicable for the structural steel.

General recommendation:

The value of Poisson's ratio may be put equal to 0.3 in the elastic state and to 0.5 in the plastic state.

:224 Bolted connections

The design of bolted connections shall be based on the characteristic values f_{buk} of the ultimate strength of the bolts in accordance with table (a) below.

Table a. Characteristic strength values for bolts

Designation [1]		f_{buk} (MPa)
Bolt	4.6	400
Bolt	8.8	800
Bolt	10.9	1 000

[1] According to SS-ISO 898-1.

:225[74] Welded connections

General recommendation:

In welded structures, special attention should be paid to the carbon equivalent.

Design of welded connections shall be based on the following:

–– For weld metal derived from standardised consumable electrodes, the characteristic strength f_{euk} shall be put equal to the ultimate strength (R_m). (BFS 1998:39)

[74] The amendment means that the reference to documents in the first item is deleted.

– For weld metal derived from consumable electrodes which are not standardised, f_{euk} shall be put equal to the nominal minimum value of the ultimate strength according to the manufacturer's documentation.

General recommendation:

The strength properties of non-standardised electrodes should be checked in accordance with Swedish Standard SS 06 01 01 or SS 06 01 11.

:226 Deviations in size and shape

In designing columns and other similar elements of structure in compression which have normal fabrication and erection tolerances, the deviations from size and shape shall be taken into consideration.

General recommendation:

These deviations should be taken into consideration as set out below unless a special investigation shows that some other method is more correct:

— The structure is assumed to have an unintended initial curvature and initial inclination in the direction of deflection under consideration.

— The initial curvature is expressed as the greatest distance e_0 between the real and theoretical system lines. The curvature is assumed to be sinusoidal or parabolic, with the deviation $e_0 = 0.0015l$ where l denotes the length of the column. For parts of a longer structure the same rules may be applied.

— The initial inclination for an element of structure which does not interact with others is assumed to be 0.005. If a number of elements interact, the initial inclination may be assumed to be smaller.

— The effect due to unintended load eccentricity may be assumed to have been allowed for by the assumption regarding initial curvature.

Values of tolerances greater than those normally specified for fabrication and erection may be applied. In such cases, correspondingly larger values shall also be used in the design assumptions.

8 :3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions regarding design are set out in Section 2:3. *(BFS 1995:18)*

8 :31 Design in the ultimate limit states

:311 Calculation of forces and moments

The analytical model shall pay special attention to the effect of the following factors unless their effect has negligible significance for the results:

- local buckling,
- flange curling and shear deformations.

If the limit state theory is applied in calculating forces and moments, the structure shall be designed so that its deformation capacity is sufficiently large for the intended distribution of forces and moments to be attained.

General recommendation:

Examples of the way in which this deformation requirement can be satisfied are given in Subclause 3:32 of *BSK 94*.

:312 Calculation of resistance

A model for calculation of the resistance shall pay special attention to the following:

- the effect of local buckling,
- the effect of flange curling and shear deformations.

General recommendation:

Examples of models for calculation of the resistance are given in Clause 3:4 of *BSK 94*.

The design values of strength, modulus of elasticity and shear modulus at the ultimate limit states shall be determined from formulae (a) - (e) below.

$$f_{yd} = \frac{f_{yk}}{\gamma_m \gamma_n} \quad \text{(a)}$$

$$f_{ud} = \frac{f_{uk}}{1.2 \gamma_m \gamma_n} \quad \text{(b)}$$

If $f_{ud} < f_{yd}$, the value of f_{ud} may be put equal to f_{yd}.

$$f_{rd} = \frac{f_{rk}}{1.1 \gamma_n} \quad \text{(c)}$$

$$E_d = \frac{E_k}{\gamma_m \gamma_n} \quad \text{(d)}$$

$$G_d = \frac{G_k}{\gamma_m \gamma_n} \quad \text{(e)}$$

NOTATION

f_{yk} characteristic value of yield strength according to Subsection 8:221

f_{uk} characteristic value of ultimate tensile strength according to Subsection 8:221

f_{rk} characteristic value of fatigue strength according to Subsection 8:222

E_k characteristic value of modulus of elasticity according to Subsection 8:223

G_k characteristic value of shear modulus according to Subsection 8:223

γ_m partial factor with regard to the uncertainty in determining the resistance.

γ_n partial factor with regard to the safety class according to Subsection 2:115

f_{yd} and f_{ud} refer to both compressive strength and tensile strength.

In the ultimate limit state the value of the partial factor γ_m shall be as follows:

a) $\gamma_m = 1.0$ if the tolerances specified on the drawings or in other documents are so tight that dimensional deviations inside these tolerances have little significance for design.

b) $\gamma_m = 1.1$ if the conditions in a) above are not satisfied.

General recommendation:

Examples of the way in which the requirements under a) can be complied with are given in Subclause 3:42 of *BSK 94*.

The value of the partial factor γ_m for design with respect to accidental action and progressive collapse is given in Subsection 2:322.

The effect of residual stresses on stiffness and resistance shall be taken into consideration.

General recommendation:

Examples of models for residual stresses are given in Clause 3:44 of *BSK 94*. The effect of residual stresses may be considered to have been allowed for in design in accordance with *BSK 94*.

:3122 Fatigue

In conjunction with fatigue action, the effects of actions shall be calculated by the elastic theory.

General recommendation:

The effect of fatigue may be taken into consideration by making a special additional calculation of the resistance with respect to fatigue, account being taken of the effect of e.g. the stress spectrum and notch action.

Design may alternatively be carried out on the basis of tests. In such a case the factor of safety with respect to fatigue failure shall be commensurate with the requirement concerning strength in Subsection 8:222.

:3123 Shell structures

General recommendation:

Examples of suitable methods for the design of shell structures are given in *Shell Handbook*, Mekanförbundets förlag, Stockholm 1990.

:3124 Bolted connections

The resistance of a bolted connection in the ultimate limit states shall be calculated for both the bolts and the parent material. In calculating the resistance, the effect of any deformations in the connection shall be taken into consideration. In a high strength friction grip connection the resistance shall also be calculated with respect to slip.

The design value of the strength of bolts in the ultimate limit states shall be determined from formula (a) below.

$$f_{\text{bud}} = \frac{f_{\text{buk}}}{\gamma_m \gamma_n} \quad \text{(a)}$$

(BFS 1998:39)

NOTATION

f_{buk} characteristic value of the ultimate strength of the bolts in accordance with Subsection 8:224

γ_m partial factor with regard to the uncertainty in determining the resistance. *(BFS 1998:39)*

γ_n partial factor with respect to the safety class in accordance with Subsection 2:115

In the ultimate limit states, the value of the partial factor γ_m shall be put equal to 1.2. *(BFS 1998:39)*

General recommendation:

Examples of suitable methods for calculating the loadbearing capacity of bolted connections are given in Clause 6:4 of *BSK 94*.

:3125 Weld

The resistance of a welded connection in the ultimate limit states shall be calculated for both the weakest section through the weld and the section immediately adjoining the weld. For a welded connection of limited length, stresses may for design purposes be assumed to be uniformly distributed over the length of the weld.

General recommendation:

Examples of suitable methods for calculating the resistance of welded connections are given in Clause 6:3 of *BSK 94*.

8:32 Design in the serviceability limit states

General recommendation:

General requirements concerning design in the serviceability limit states are set out in Section 2:12.

If plastic deformations occur their effect shall be taken into consideration, but residual stresses may be ignored.

In the serviceability limit states, the design values may be put equal to the appropriate characteristic values.

General recommendation:

Design in the serviceability limit states should be in accordance with the elastic theory, with the analytical model in accordance with Section 8:31 as appropriate.

8:33 Design by testing *(BFS 1995:18)*

In the ultimate limit states, the design value R_d shall be determined from formula (a) below.

$$R_d = \frac{R_k}{\gamma_{mp}\,\gamma_n} \qquad \text{(a)}$$

NOTATION

R_k characteristic value of resistance in accordance with Subsection 2:33

γ_{mp} partial factor, in accordance with Table (a) below, which takes account of the uncertainty in determining the resistance

γ_n partial factor which takes account of safety class in accordance with Section 2:1

Table a. Partial factor γ_{mp} in the ultimate limit states

Type of failure	γ_{mp}	
Yield or instability failure	1.15	(1.05)[1]
Failure in material	1.3	(1.2)[1]
Failure in welded or bolted connection	1.2	
Fatigue failure	1.1	

[1] The values in brackets apply for small tolerances in accordance with Subsection 8:312, i.e. in cases for which $\gamma_m = 1.0$.

General recommendation:

In the serviceability limit states the design values can be determined in accordance with Boverket's handbook *Structural design by testing*.

8:4 Materials

General recommendation:

General provisions regarding materials are given in Subsection 2:4.

8:41 Bolted connections

Fasteners (bolts, nuts, washers and threaded structural elements) shall have documented strength.

The properties of bolts and associated nuts for preloaded bolted connections shall be such that the nuts and threads are normally stronger than the bolts even in conjunction with unfavourable combinations of properties and sizes. In other connections the strength of the nut shall be not less than the nominal ultimate tensile strength of the bolt.

General recommendation:

Examples of fasteners are given in Clause 7:14 of *BSK 94*.

8:42 Filler metal

The properties of filler metal for welding shall be such that a welded connection has the intended function and durability. The strength and other essential material properties shall be documented.

The filler metal shall be suited to the welding process, the parent material, the welding procedure and the requirements specified for the welded connection.

Where there is a risk of hydrogen cracking, the filler metal used shall be such as to give rise to a low hydrogen content in the weld metal.

For structures in environments of high or very high aggressivity, the filler metal used shall produce a weld metal which has at least the same corrosion resistance as the parent material.

General recommendation:

Examples of filler metal are given in Clause 7:13 of *BSK 94*.

8:43[75] Properties in the thickness direction

In structures acted upon by tensile forces in the thickness direction, action shall be taken to ensure that transmission of force in the thickness direction is satisfactory in view of the risk of laminar tearing in the steel.

General recommendation:

Action to ensure that transmission of force in the thickness direction is satisfactory should be suited to the safety class and degree of utilisation of the structure and its design. It may, for example, consist of the selection of material of guaranteed and verified properties in the thickness direction in accordance with Subclause 7:22 of *BSK 94*. *(BFS 1998:39)*

8:5 Execution and workmanship

General recommendation:

General provisions regarding execution and workmanship are given in Section 2:5. Examples of suitable execution and workmanship are given in Section 8 of *BSK 94*. *(BFS 1995:18)*

8:51 Handling of materials

Plates, sections, hollow sections, filler metal for welding, fasteners and similar shall be stored and handled in such a way that different materials cannot be confused, and so that the intended properties do not deteriorate to a harmful extent.

Marking shall be such that the relationship between the material and the associated certificate is secured and confusion is prevented.

8:52 Preparation of materials

General recommendation:

In conjunction with the preparation of materials, Clause 8:3 of *BSK 94* should be taken into consideration.

8:53 Welded connections

:531 Welding

Welding in a steel structure shall be carried out only where welding is specified on the drawings.

General recommendation:

Examples of guidelines regarding welding are given in Swedish Standards SS 06 40 01 and SS 06 40 25.

The regulations and general recommendations of the Swedish Board of Occupational Safety and Health regarding fusion welding and thermal cutting are set out in *AFS 1992:9*.

[75] Latest wording BFS 1995:18.

:532 Welding procedure sheet

A welding procedure sheet shall be drawn up for welding. An exception may be made for simple work of routine character.

General recommendation:

The person who directs and supervises welding should draw up the welding procedure sheet in consultation with the designer. *(BFS 1995:18)*

In conjunction with welding of a complicated nature for which practical experience is not available, welding tests should be carried out before the welding procedure sheet is drawn up.

Examples of what a welding procedure sheet should contain are given in Clause 1:42 of *BSK 94*.

:533 Welders' qualifications

General recommendation:

General provisions regarding workmanship are given in Section 2:5.

An approved welder's test in accordance with EN 287-1:1992 may be considered to be an example of documented qualifications.

8:54 Bolted connections

:541 Holes for bolts and matching of holes

Holes shall be made by a method which provides sufficient accuracy with regard to the size and placing of the hole, and in such a way that the strength and ductility of the parent material do not deteriorate to a deleterious extent.

General recommendation:

If the displacement between holes in parts of the same connection is excessive, the holes may be drilled or reamed to the next larger bolt diameter, due attention being paid to the appropriate requirements regarding matching of holes.

Examples of methods which comply with the requirements of the mandatory provision are given in Subclauses 8:511 and 8:512 of *BSK 94*.

Examples of tolerances are given in Publication No 112, 1992, *Tolerances for steel structures*, of the Swedish Institute of Steel Construction.

:542 Contact surfaces

Contact surfaces in bolted connections shall fit together so that the contact necessary for the function of the connection is achieved.

General recommendation:

Special attention should be paid to fit in preloaded connections in order to prevent losses of clamping force.

Examples of the classification of contact surfaces and the treatment of contact surfaces are given in Clause 8:5 of *BSK 94*.

:543 Assembly and securing of bolted connections

In preloaded connections each bolt shall be preloaded to not less than 70% of the nominal ultimate tensile strength of the bolt in order that the specified clamping force should be attained. Nuts shall be specially secured where the direction of the force on the bolt alternates.

General recommendation:

Connections in other classes should be tightened normally and the nuts reliably secured.

In close tolerance connections the unthreaded shank of the bolts should normally terminate outside the parent material.

In preloaded connections washers should be used if without a washer the local pressure under the bolt head and nut due to the preloading force exceeds the design value of the ultimate strength of the parent material.

A bolt which has been preloaded and thereafter undone should be rejected and replaced.

Examples of appropriate assembly of bolted connections are given in Subclauses 8:53 and 8:54 of *BSK 94.*

8:55 Dimensional accuracy in fabrication and erection

Steel structures shall be erected with the intended accuracy with respect to dimensions and shape.

The deviations from shape in the finished structure shall not exceed the tolerances assumed in design.

If a compressive force is assumed to be transmitted in direct bearing by the contact surfaces of two parts in a welded structure, the parts shall be fabricated so that the contact surfaces have the required fit.

General recommendation:

Examples of tolerances for fabrication and erection are given in Publication No 112, 1992, *Tolerances for steel structures,* of the Swedish Institute of Steel Construction.

8:56 Corrosion protection

Steel structures in a corrosive environment shall be provided with the necessary corrosion protection.

General recommendation:

Corrosion protection may be in the form of an appropriate coating, cathodic protection or corrosion allowance.

Examples of the classification of corrosive environments and appropriate methods of providing corrosion protection are given in Subclause 1:23 and Clause 8:7 of *BSK 94*.

The requirements concerning technical and personnel conditions in conjunction with corrosion protection painting may be considered to have been complied with if the firm in question has been approved by the *Authorisation Board for Corrosion Protection Painting.*

8:57 Erection

Erection shall be carried out in accordance with an erection schedule. Work shall not begin until an erection schedule is available.

General recommendation:

The person who directs and supervises erection should draw up the erection schedule in consultation with the designer.
(BFS 1995:18)

Examples of what an erection schedule should contain are given in Subclause 1:43 of *BSK 94*.

8:6 Supervision and control

The values of the partial factors γ_m and γ_{mp} set out in this section presuppose that the supervision and control specified in Section 2:6 is carried out. *(BFS 1995:18)*

General recommendation:

For structures in safety class 1 the scope of inspections and tests may be assessed in view of circumstances. *(BFS 1995:18)*

Acceptance inspection on delivery, apart from identification and visual examination, may be carried out as spot checks if the materials or products delivered are likely to comply with the stipulated requirements, e.g. due to the availability of acceptance certificates from previous tests. Acceptance certificate Type 3.2 in accordance with SS-EN 10 204 should be used.

8:61 Basic inspection

General recommendation:

Basic inspection should comprise the following areas:

– materials,
– dimensions and shape,
– welded connections,

– bolted connections, and
– surface finish for fire and corrosion protection. *(BFS 1995:18)*

Examples of the measures to be taken in conjunction with basic inspection are given in Section 9 of *BSK 94*.

If the scope of basic inspection is not specified in detail, it may be confined to sampling inspection. In such a case the inspection measures should have a scope such that there is a satisfactory degree of certainty that the structure as a whole complies with the requirements.

8:62 Additional inspection

General recommendation:

Additional inspection should comprise the following areas:
– inspection measures relating to the specific structure in question,
– elements of structure subject to tensile force in the thickness direction,
– welded connections, and
– cathodic protection. *(BFS 1995:18)*

Examples of inspection measures in conjunction with additional inspection are given in Section 9 of *BSK 94*.

9 ALUMINIUM STRUCTURES

The provisions of this section relate to loadbearing structures of aluminium sheeting and extruded aluminium members.

General recommendation:

Aluminium structures of cold formed sheeting designed, constructed and inspected in accordance with StBK-N5, *Swedish Code for Thin Gauge Structures 79*, comply with the requirements for loadbearing structures set out in Section 2.

9:1 Requirements

The provisions of Section 8:1 also apply as appropriate for aluminium structures.

General recommendation:

General requirements are set out in Section 2:1.

9:2 Design assumptions

General recommendation:

General design assumptions are set out in Section 2:2.

9:21 Actions

The provisions of Subsection 8:21 apply as appropriate for aluminium structures.

9:22 Characteristic values

:221 Strength, modulus of elasticity and shear modulus

For standardised grades of aluminium, the characteristic values of the strength of aluminium, f_{yk} for the 0.2% proof strength and f_{uk} for the ultimate tensile strength, shall be as set out in Table (a) overleaf.

For other grades of aluminium, the values of f_{yk} and f_{uk} shall be determined individually for each grade and condition on the basis of tests relating to the 0.2% proof strength and the ultimate tensile strength.

The characteristic values apply for structures with a service temperature lower than 60°C.

The values of the modulus of elasticity and shear modulus shall be assumed to be E_k = 70 GPa and G_k = 27 GPa.

Table a. Characteristic values of mechanical properties for standardised grades of aluminium

Alloy[1]		Condition[2]	Characteristic strength (MPa)			Elongation at failure
SS[3]	ISO[4]		f_{uk}	f_{yk}	f_{wuk}	A_5 (%)
4007	1050A	-00 Hot worked	65	30	65	30
		-14 Cold worked	110	90	75	6
		-18 Cold worked	150	130	75	3
4054	3103	-14 Cold worked	140	115	95	6
		-18 Cold worked	185	165	95	3
4103	6060	-06 Artificially aged	190	150	100	10
4104	6063	-06 Artificially aged	210	170	100	12
4107	6005	-06 Artificially aged	260	225	—	8
	6005A	-06 Artificially aged	260	225	160	8
4115	5049	-00 Hot worked	190	80	190	10
		-02 Annealed	190	80	190	18
		-14 Cold worked	240	190	190	5
		-18 Cold worked	290	250	190	3
4120	5052	-00 Hot worked	190	75	170	10
		-02 Annealed	170	65	170	20
		-14 Cold worked	210	160	170	12
		-18 Cold worked	280	240	170	3
		-24 Partially annealed	220	170	170	14
4125	5754	-00 Hot worked	190	80	190	10
		-02 Annealed	190	80	190	18
		-16 Cold worked	265	215	190	4
		-18 Cold worked	290	250	190	3
		-26 Partially annealed	265	190	190	7
4140	5083	-00 Hot worked	275	125	270	12
		-02 Annealed	270	120	270	17
		-12 Cold worked	310	205	270	12
		-22 Partially annealed	310	205	270	13
		-24 Partially annealed	345	270	270	6
4212	6082	-04 Naturally aged	205	115	170	15
		-06 Artificially aged	290	245	180	8
4338	2014	-04 Naturally aged	350	230	—	14
		-06 Artificially aged	430	380	—	4
4425	7020	-04 Naturally aged	275	145	260	12
		-06 Artificially aged	340	270	260	7

(BFS 1998:39)

1 Aluminium alloys for cold formed sheeting are given in *StBK-N5, Swedish Code for Thin Gauge Structures 79.*

2 The materials in condition -04 and -06 are heat treatable, the others are not heat treatable.

3 Swedish Standard in accordance with MNC 40.

4 In accordance with ISO 209-1.

9:3 Design by calculation and testing

(BFS 1995:18)

General recommendation:

General provisions regarding design are set out in Section 2:3. *(BFS 1995:18)*

9:31 Design in the ultimate limit states

:311 Calculation of forces and moments

For the limit state theory to be applied, the characteristic value of the elongation at failure, A_5, shall be greater than 6%, and the characteristic value of the ratio of the ultimate tensile strength f_{uk} to the yield strength f_{yk} shall be not less than 1.10.

General recommendation:

The stress-strain curve may conform to formula (a) below:

$$e = \frac{s}{E_d} + 0.0002\left(\frac{s}{f_{yd}}\right)^n \leq 0.6A_5 \qquad \text{(a)}$$

where $n = f_{yk}/10$ (with f_{yk} in MPa).

:312 Calculation of resistance

:3121 Design

Design values in the ultimate limit states shall be determined from formulae (a) - (c) below.

$$f_{yd} = \frac{f_{yk}}{\gamma_m \gamma_n} \qquad \text{(a)}$$

(BFS 1995:18)

$$f_{ud} = \frac{f_{uk}}{1.2\gamma_m \gamma_n} \qquad \text{(b)}$$

$$E_d = \frac{E_k}{\gamma_m \gamma_n} \qquad \text{(c)}$$

where f_{yd} and f_{ud} refer to both compressive strength and tensile strength.

If f_{ud} according to formula (b) is smaller than f_{yd} according to formula (a), f_{ud} may be put equal to f_{yd}.

The value of γ_m shall be put equal to 1.0 on condition that the resistance is determined with reference to a reduced cross section based on the lower limit of size (basic size less the lower limit of deviation), otherwise its value shall be 1.1.

The value of γ_n depends on the safety class and shall be determined in accordance with Subsection 2:115.

:3122 Local buckling

General recommendation:

Extruded aluminium members should be assigned to the same category as hot formed steel members.

:3123 Compressive force

General recommendation:

The reduction factor ω_c for flexural buckling may be determined in accordance with Subclause 6:23 of *BSK 94*.

Extruded members should be classified on the basis of material and type of cross section in accordance with Table (a) below. The classification of members with longitudinal welds should be the same as that of welded steel members. In addition, reduction of strength in the heat affected zone should be taken into consideration in accordance with Subsection 9:3125.

Table b. Material, type of cross section and group for the determination of w_c

Material	Type of cross section	Group
Heat treatable	Symmetrical	a
Non-heat treatable	Symmetrical	b
Heat treatable	Non-symmetrical	c
Non-heat treatable	Non-symmetrical	d

:3124 Bending moment

General recommendation:

In determining the shape factor η in accordance with the Handbook Bygg, Section K18:42, the plastic moment of resistance Z should be multiplied by

$$m_z = 0.6 + 0.33\,\frac{f_{uk}}{f_{yk}} \qquad \text{(a)}$$

:3125 Welded connections

General recommendation:

Welded connections should be designed in accordance with the methods set out in *BSK 94*, with the characteristic values for weld material equal to f_{wuk} according to Table (a) in Subsection 9:22.

For longitudinally welded members and beams, the reduction in strength in the heat affected zone HAZ is to be taken into consideration by assuming that, within a region 25 mm on each side of the weld, the value of the effective thickness is given by formulae (a) and (b) below:

$$t_{\mathrm{HAZ}} = \frac{f_{\mathrm{wud}}}{f_{\mathrm{ud}}} t \qquad \text{(a)}$$

$$f_{\mathrm{wud}} = \frac{f_{\mathrm{wuk}}}{1.2\gamma_{\mathrm{m}}\gamma_{\mathrm{n}}} \qquad \text{(b)}$$

NOTATION

f_{wuk} characteristic value for material in the heat affected zone according to Table (a) in Subsection 9:22

f_{ud} design value of strength for the parent material

t material thickness

If the thickness is also reduced with respect to local buckling (t_{ef}), the effective thickness is to be put equal to the lesser of t_{ef} and t_{HAZ}.

For members and beams with transverse welds (welded splices or attachment of stiffeners etc), f_{yd} is to be replaced by f_{wud} if $f_{\mathrm{wud}} < f_{\mathrm{yd}}$ within a region 25 mm on each side of the weld.

:3126 Bolted conn

General recommendation:

In Equation 6:432b in *BSK 94*, the value of e_1 may be put equal to 2d if $e_1 > 2d$.

The value of the coefficient of friction μ_k between clean aluminium surfaces should be not greater than 0.3.

:3127 F

General recommendation:

The characteristic fatigue strength f_{rk} (MPa) should be determined in accordance with *European Recommendations for the Design of Aluminium Alloy Structures*, European Convention for Constructional Steelwork Publication No 68, 1992.

Partial factors should be selected in accordance with Clause 6:5 of *BSK 94*.

9:32 Design in the serviceability limit states

The provisions of Subsection 8:32 also apply as appropriate for aluminium structures.

9:33 Design by testing (*BFS 1995:18)*

The provisions of Subsection 8:33 also apply as appropriate for aluminium structures.

9:4 Materials

The provisions of Section 8:4 also apply as appropriate for aluminium structures.

9:5 Execution and workmanship

The provisions of Section 8:5 also apply as appropriate for aluminium structures.

9:6 Supervision and control

The provisions of Section 8:6 also apply as appropriate for aluminium structures.

10 RESISTANCE IN CASE OF FIRE

Further mandatory provisions and general recommendations regarding the resistance of buildings in case of fire are to be found in Section 5:8 of Boverket's Building Regulations, BBR.
(BFS 1998:39)

10:1 Requirements

Parts of the loadbearing structure, inclusive of supports, joints, connections and similar, shall be constructed in such a way that collapse does not occur either
— within a certain period of time according to the requirements applicable to the fire resistance classes specified for elements of structure in Subsection 5:82 of BBR,
— during a complete fire process, or
— during part of a complete fire process, if it can be shown by a special investigation that the safety of escape is not affected adversely and that the risks for the personnel of the rescue service and the effects on the environment are not increased.
(BFS 1998:39)

General recommendation:

In the same way as in conjunction with ordinary combinations of action, the requirements regarding safety against failure in case of fire should be differentiated in view of the consequences of failure. The factors which influence the choice of safety class in an ordinary combination of actions, namely the type and use of the building, the type of the loadbearing structure or element of structure and the character of the envisaged failure, are also relevant in the event of fire. In a fire, the consequences of failure are to a high degree dependent on whether there are still people inside the building when failure occurs. This implies that the longer the period of time after the outbreak of fire during which there is a certain probability that people are present in the building or in its immediate vicinity, the more stringent should be the requirements regarding structural safety.

In *design by classification* in accordance with Subsection 5:82 of BBR, these conditions are taken into consideration by the fire resistance class prescribed for the application in question; this class is dependent on the use of the building, the height of the building, the magnitude of the fire load density, and the significance of the element of structure for the overall resistance of the building structure. *(BFS 1998:39)*

In *design based on a model of a parametric fire exposure* in accordance with Section 5:83 of BBR, the above conditions are taken into consideration by differentiating the design fire load density and the duration of the fire with regard to the application in question. In this way, the influence of the factors which affect the selection of safety class for the design resistance of the building structure in the event of fire is taken into consideration indirectly. *(BFS 1998:39)*

During a fire, considerable temperature movements may occur in the loadbearing structure of the building. For frames and other statically indeterminate structures, these movements may give rise to appreciable increments to, and redistributions of, section forces and section moments, and cause cracking and other damage in e.g. columns, beams, floor constructions and walls. These effects occur not only in the elements of structure directly affected by fire but also in the building carcass outside the fire compartment in question. It is essential that these effects should be taken into consideration in design, and that the building carcass should be detailed appropriately with regard to these effects.

10:11 Factor of safety with respect to failure and instability in case of fire

The partial factor γ_n may be put equal to 1.0 irrespective of the safety class of the structure.

The design load effect S_d shall be determined for the most unfavourable load combination, using the partial factor γ_f for load in accordance with Table (b) in Subsection 2:321.

The design resistance R_d according to the method of partial factors shall be determined in view of the following conditions:

— Consideration shall be given to the reduction in strength at elevated temperatures and to the reductions in effective cross section due to combustion and the action of fire. In calculations, the strength and deformation properties, thermal conductivity and specific heat capacity of each material must be sufficiently well known within the temperature region concerned.

— Consideration shall be given to the changes in the properties of fasteners, connectors and similar under the action of fire.

— The value of the partial factor γ_m for materials in accordance with Subsection 2:322 may be assumed to be equal to 1.0 unless other values are specified in Sections 4 - 9.

10:2 Design by calculation and testing

(BFS 1995:18)

10:21 Determination of resistance by classification

The characteristic resistance of a loadbearing element of structure may be determined by *testing* in accordance with Swedish Standard SIS 02 48 20 (Nordic Standard NT FIRE 005, ISO 834). The element of structure is assumed to be acted upon by an external static load during the entire test period, corresponding to the intended period of fire resistance.

This load shall be adjusted so that the stresses at critical sections are the same as those which occur due to the design loads in the event of fire in accordance with Subsection 2:321. Temperature development at critical sections shall if possible be recorded during the test.

The resistance of the structure for a certain period of fire resistance shall be determined on the basis of associated values of applied action and time.

The characteristic resistance of a structure may be *calculated* on the basis of the conditions set out in Section 10:11 and the fire exposure in accordance with SIS 02 48 20 (NT FIRE 005, ISO 834). The assumptions regarding dimensions, spans, support conditions, design in other respects and mechanical moduli shall be made in accordance with the principles which are approved in design without regard to fire in accordance with Section 2.

The characteristic resistance of a loadbearing structure in the event of fire may be determined by *combined testing and calculation.* The tests may be made on unloaded test objects if loading cannot be assumed to affect the behaviour of the test object. Temperature development at critical sections shall if possible be recorded during the test. On the basis of the recorded temperature curves and e.g. the measured depth of fire penetration in timber structures, the resistance can then be calculated if the relevant material data are known and verified.

10:22 Determination of resistance by design based on a model of a parametric fire exposure

Determination of the resistance of the structure on the basis of a model of a parametric fire exposure can in certain cases be made by testing. A combination of testing and calculation may also be appied. In all cases, the mandatory provisions of Section 10:21 shall apply as appropriate.

:221 Fire load density

The design value of the fire load density shall be the value which is included in 80% of the observed values in a representative statistical material. However, in designing elements of structure which, according to Column 1 of Table (a) in Subsection 5:821 of BBR, shall be constructed to Class R 90, this value of the fire load density shall be increased by 50%. *(BFS 1998:39)*

Elements of structure which shall be constructed to Class R 60 or higher shall be designed for a complete fire process (inclusive of cooling), while for lower fire resistance classes design shall be based on the time indicated by the numerical value of the class designation (but exclusive of cooling).

General recommendation:

Examples of characteristic values are given in the report *Fire engineering design of concrete structures,* published by the Swedish Council for Building Research, 1992.

:222 Fire compartment temperature

The gas temperature T_t in a fire compartment is to be calculated from heat and mass balance equations (model of a parametric fire exposure). Consideration may be given to an automatic water sprinkler installation and fire gas ventilation.

Where flashover is not likely to occur and the fire will be limited in extent, the gas temperature T_t may be assumed to depend on the area and heat output of the fire, and not on the magnitude of the fire load density.