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Steel structures —

Part 1:
Materials and design

Structures en acier —

Partie 1: Matériaux et conception

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

International Standard ISO 10721-1 was prepared by Technical Committee ISO/TC 167, *Steel and aluminium structures*, Subcommittee SC 1, *Steel: Material and design*.

ISO 10721 consists of the following parts under the general title *Steel and aluminium structures*:

— Part 1: Materials and design

— Part 2: Fabrication and erection

Annexes A and B of this part of ISO 10721 are for information only.

Introduction

This part of ISO 10721 establishes a common basis for drafting national standards for the use of materials in steel structures and for their design, in order to ensure adequate and consistent measures regarding safety and serviceability.

Annex A of this part of ISO 10721 contains noncompulsory recommendations which may be used as guidelines for practical design.

The specific and numerical requirements for the completion of structures which are optimal with respect to the state of a country's economy, development and general values should be given in the national codes of the country.

The design rules given concern limit-state verifications for comparing the effects of actions or combinations of actions with the strength (resistance) of the structure and its components.

Steel structures —

Part 1:
Materials and design

1 Scope

This part of ISO 10721 establishes the principles and general rules for the use of steel materials and design of steel structures in buildings.

NOTE 1 The degree of reliability should be as specified in national codes. In the establishment of design safety factors, due consideration should also be given to ISO 10721-2 for fabrication of steel structures.

This part of ISO 10721 is also applicable to bridges, off-shore and other civil engineering and related structures, but for such structures it may be necessary to consider other requirements.

This part of ISO 10721 does not cover the special requirements for steel structures in corrosive environments beyond normal atmospheric conditions and corrosion protection with regard to fatigue design.

This part of ISO 10721 does not cover the special requirements of seismic design.

For welded connections and for structures subject to fatigue, special considerations regarding the scope of this document are presented in 8.9 and 10.1 respectively.

NOTE 2 Rules and recommendations regarding composite steel and concrete structures and fire safety of steel structures will subsequently be issued as separate International Standards.

2 Normative references

The following standards contain provisions which, through reference in this text, constitute provisions of this part of ISO 10721. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this part of ISO 10721 are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below. Members of IEC and ISO maintain registers of currently valid International Standards.

ISO 630:1995, *Structural steel — Plates, wide flats, bars, sections and profiles.*

ISO 898:1988–1994, *Mechanical properties of fasteners* (all parts).

ISO 2394: –[1], *General principles on reliability of structures.*

ISO 3989: –[2], *Bases for design of structures — Notations — General symbols.*

ISO 4753:1983, *Fasteners — Ends of parts with external metric ISO thread.*

ISO 4951:1979, *High yield strength steel bars and sections*

ISO 6892: –[3], *Metallic materials — Tensile testing at ambient temperature.*

1) To be published. (Revision of ISO 2394:1986)

2) To be published. (Revision of ISO 3898:1987)

3) To be published. (Revision of ISO 6892:1984, replacing ISO 82:1974)

3 DEFINITIONS AND SYMBOLS

For the purposes of this part of ISO 10721, the following definitions and symbols apply.

3.1 Definitions

Limit states:	The states beyond which the structure no longer satisfies the design requirements.
Ultimate limit state:	The limit states corresponding to the maximum load carrying resistance (safety related).
Serviceability limit state:	The limit states related to normal use (often related to function).
Specified life:	The time the structure is to be used under the given design assumptions.
Direct action:	One or a set of concentrated or distributed forces acting on the structure, such as selfweight, imposed specified actions, wind, etc.
Indirect action:	The cause of imposed or constrained deformations in the structure, such as temperature effects, settlements, creep etc.
Nominal action:	The numerical value of an action either defined by the authorities or by the contract documents. When this value corresponds to a specified probability to be exceeded within a specified reference time, it is called characteristic action, and it is calculated in accordance with ISO 2394.
Design action:	Actions used in calculations. The design action is the nominal action multiplied by its partial safety factor γ_f, or it is the combination of nominal actions, each multiplied by its partial safety factor γ_f for the relevant limit state.

Shake down:	The process of local yielding due to the initial applications of variable actions, leading to a condition of residual stress where all further applications can be sustained elastically (applies particularly to the formation of plastic hinges).
Variable action:	Action which is unlikely to act throughout a given design situation or for which the variation in magnitude with time is not monotonic nor negligible in relation to the mean value.
Repetetive action:	Design action which involves stress fluctuations leading to possible fatigue effects, i.e. it is the design action to be used for checking the fatique limit state.
Characteristic material property:	The value of material properties established by its specified occurrence taking account of control conditions and statistical variability.
Design material property:	The value of material properties obtained by dividing the characteristic property by a partial material safety factor.
Nominal strength or resistance:	The strength or resistance value based on specified characteristic material and geometric properties.
Design strength or resistance:	The nominal strength or resistance divided by the appropriate partial safety factor for resistance, γ_r.
Normal use:	Normal use is that which conforms to the loading and performance intended by the designer, or as specified in codes of practice, or by other relevant requirements.
Fatigue:	Damage, by gradual crack propagation in a stuctural part, caused by repeated stress fluctuations.
Fatigue loading:	A set of typical load events described by the position of loads, their intensities and their relative occurence.
Loading event:	A defined loading sequence applied to the structure and giving rise to a stress history variation.
Equivalent fatigue loading:	A simplified fatigue loading representing the fatigue effects of all loadings events.
Stress history:	A record or a calculation of the stress variation at a particular point of a structure during the load event.
Stress range:	The algebraic difference between two extrema of the stress history ($\Delta\sigma s = \sigma_{max} - \sigma_{min}$ or $\Delta\tau = \tau_{max} - \tau_{min}$). This difference is usually identified by a stress cycle counting method.
Nominal stress:	A fatigue design stress in the parent material adjacent to potential crack location calculated in accordance to simple elastic strength of materials theory. For the purpose of fatigue assessment of a particular class of constructional detail, the design stress is either the normal stress (axial and bending stress) or/and the shear stress. Where there is a geometric discontinuity, not taken into account in the classification of the constructional detail, the nominal stress shall be modified by the use of stress concentration factors.

Geometric stress:	A fatigue design stress, adjacent to the weld toe, defined as the extrapolation of the maximum principal stresses. The geometric stress takes into account the overall geometry of the constructional detail, excluding local stress concentration effects due to weld geometry and inherent defects in weld and adjacent parent metal. (The geometric stress is often referred in the litterature as the "hot spot stress").
Cycle counting:	A particular method used for counting the number of stress cycles and related stress ranges from a stress history.
Stress-range spectrum:	Histogram of the frequency of occurrence for all stress ranges of different magnitudes recorded or calculated for a particular loading event.
Design spectrum:	The total of all stress spectra relevant to the fatigue assessment.
Equivalent stress range:	The constant-amplitude stress range that would result in the same fatigue life (number of cycles of stress ranges) as for the spectrum of variable amplitude stress ranges based on a Miner's summation.
Miner's summation:	A cumulative linear damage calculation based on the Palmgren-Miner rule.
Constant amplitude fatigue limit:	The limiting stress range value above which a fatigue assessment is necessary.
Detail category:	The designation given to a particular welded or bolted detail, in order to indicate which fatigue strength curve is applicable for the fatigue assessment.
Fatigue strength curve:	The quantitative relationship between stress range and number of stress cycles to fatigue failure (selected on the basis of a statistical analysis of available test data of a constructional detail).
Design life:	The reference period of time for which a structure is required to perform safely with an acceptable probability that failure by fatigue or cracking will not occur.
Cut-off limit:	Limit below which stress ranges of the design spectrum do not contribute to fatigue damage.
Groove (butt) weld:	A weld made in a preparation to receive weld metal. (Also referred to as a butt weld).

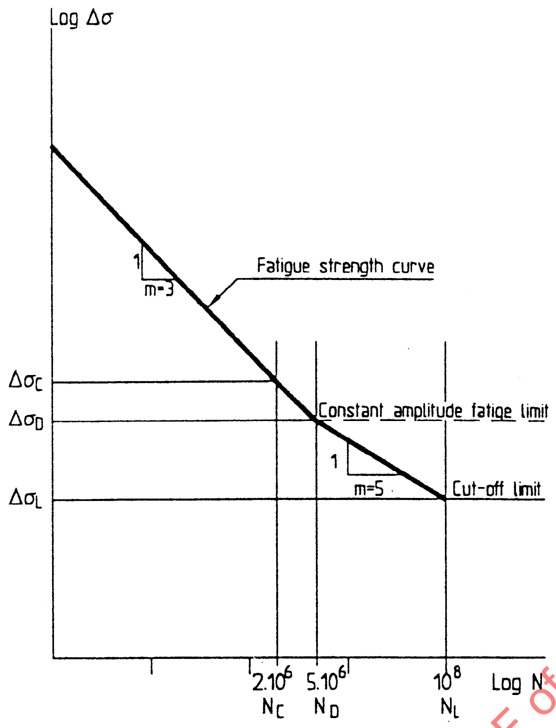


Fig. 3.1 Fatigue strength curve definitions

3.2 List of symbols
(see also ISO 3898)

LATIN UPPER CASE LETTERS:

A	Cross-sectional area
A_e	Effective cross-section area
A_g	Gross section area
A_L	Cross-sectional area of longitudinal stiffener
A_{Le}	Effective area of longitudinal stiffener
A_m	For fillet welds, A_m = effective size multiplied by its length. For butt joints, A_m = thickness of base metal multiplied by its length. For T-joints, A_m = size of fusion face in base metal multiplied by the length of the weld
A_n	Net section area
A_s	Cross-sectional area of a stiffener
A_{sp}	Nominal area of the threaded part of a bolt
A_t	Cross-sectional area of transverse stiffener
A_v	Effective shear area of bolts
A_w	Cross-sectional area of web
A_W	Effective area of weld (effective throat of weld multiplied by its length). For plug or slot welds, A_w = area of faying surface

B	Coefficient
C_w	Warping constant of torsion for the cross-section
E	Modulus of elasticity (Young's modulus)
E_T	Tangent modulus
F	Force, action
F_b	Bearing resistance of bolts
F_k	Characteristic action
F_d	Design force, action
F_p	Preloading force in bolts
F_s	Slip resistance of bolts
F_t	Tensile force resistance of bolts
F_v	Shear force resistance of bolts
G	Modulus of shear $= E/(2(1+\upsilon))$
I, I_y, I_z	Moment of inertia (about y- and z-axis, respectively)
I_s	Moment of inertia of a stiffener
I_p	Polar moment of inertia
I_t	St. Venant torsion constant of the cross-section
K_E	Coefficient for buckling length
K_i	Coefficients (i = 1-5)
L	Length
L_E	Effective length (in buckling)
L_o	Laterally unsupported length
L_s	Load distribution length
M, M_y, M_z	Bending moment (about y- and z-axis, respectively)
M_{rd}, M_{dy}, M_{dz}	Moment resistance (about y and z-axis, respectively)
M_{dr}	Reduced moment resistance
M_{EL}	Elastic lateral torsional buckling moment
M_f	Plastic moment of flanges
M_{Ld}	Moment resistance in lateral torsional buckling
M_p	Plastic moment $= f_y W_p$

M_T	Torsional moment
M_w	Plastic moment of web
M_y	Yield moment $= f_y W$
M_1, M_2	The larger and the smaller moments at the supported ends of a member
N	Normal force (chapt. 8)
N	Number of fatigue strength cycles (chapt 10)
N_{cd}	Buckling resistance (chapt. 8)
N_c	Number of cycles ($2 \cdot 10^6$) at which the reference value of the fatigue strength curve is defined (chapt. 10)
N_{cy}, N_{cz}	Buckling resistance about y- and z- axis, respectively
N_D	Number of cycles for which the constant amplitude fatigue limit is defined ($= 5 \cdot 10^6$)
N_r	Normal force resistance
N_{rd}	Normal force design resistance
N_E, N_{Ey}, N_{Ez}	Elastic buckling force of a pinned column $= \dfrac{\pi^2 EI}{L^2}$ (about the y- and z-axis, respectively).
N_{Ecr}	Elastic buckling load of a structure $= \dfrac{\pi^2 EI}{L_k^2}$
N_{ET}	Elastic torsional buckling load
N_i	Number of cycles of stress $\Delta\sigma_i$ to cause failure
N_L	Number of cycles for which the cut-off limit is defined ($=10^8$)
N_p	Plastic normal force resistance
N_{Td}	Torsional buckling resistance
N_{yd}	Ultimate tensile yield force resistance
N_{ud}	Ultimate tensile strength
P	Concentrated force
P_d	Concentrated force resistance
R	Resistance
S	Static moment of area
T	Tensile force (in a bolt)
V	Shear force
V_{cd}	Shear resistance

V_i	Notional shear force in built up members
W, W_y, W_z	Elastic section modulus
W_e, W_{ex}, W_{ey}	Elastic section modulus of the effective cross section
W_p, W_{py}, W_{pz}	Plastic section modulus

LATIN LOWER CASE LETTERS:

a, a_1, a_2	Distance. The weld throat "a-dimension"
b	Width (of plates)
b_e, b_{e1}, b_{e2}	Effective width
c	Distance
c_L, c_t	Coefficients for stiffeners
d	Diameter. Depth
d_e	Effective depth
e, e_y, e_z	Eccentricity. Distance (for bolts)
f_b	Bearing strength in bolted connections
f_c, f_{cy}, f_{cz}	Buckling strength (about y- and z-axis, respectively)
f_{cd}	f_c/γ_r
f_{cL}	Lateral torsional buckling strength
f_{cT}	Torsional buckling strength
f_d	Design strength
f_{cp}	Local elastic plate buckling strength
f_u	Specified ultimate tensile strength of base material or bolt material
f_{uw}	Specified ultimate tensile strength of weld material
f_y	Specified yield strength of material or the stress giving 0.2 % permanent strain
f_{ye}	Reduced (effective) yield strength of material
f_{yw}	Specified yield strength of weld material
g	Distance between bolt holes (the gauge)
h	Height (of web)
h_e	Effective heigth
i, i_y, i_z	Radius of gyration (about y- and z-axis, respectively)

i_p	Polar radius of gyration
k, k_c	Coefficient for the effect of the stress distribution and the support conditions on elastic plate buckling
k_y, k_z	Compression member buckling coefficients
k_s	Elastic shear buckling coefficient
l	Length, span
m	Slope constant of the fatigue strength curve. The curves have slopes of -1/3 and /or -1/5 and the corresponding values of the slope constant m are 3 and 5
n	Number. Coefficient (for built-up-members)
n_E	Equivalent number of stress cycles
n_i	Number of applied stress cycles $\Delta\sigma_i$
r	Radius
s	Distance between bolt holes (the staggered pitch). Weld size for T-welds
t	Thickness
t_f	Flange thickness
t_w	Web thickness
x, y, z	Cartesian Coordinates (x along member axis)
y_s, z_s	Shear center coordinates

GREEK LETTERS:

α	Angle. Buckling curve designation. Coefficient for arbitary eccentricity of column load. Bearing stress coefficient for bolted connections. Aspect ratio for plates
β	Coefficient for arbitrary eccentricity. Reduction coefficient for the length of bolted connections
β, β_y, β_z	Equivalent uniform moment coefficient for beam-columns
γ	Partial coefficient
γ_f	Partial safety factor for actions
γ_r	Partial safety factor for resistance (in this document identified as resistance factor)
γ_{rc}	Resistance factor for a connection
γ_{rs}	Slip resistance factor

$\Delta\sigma$	Nominal stress range (normal stress)
$\Delta\tau$	Nominal stress range (shear stress)
$\Delta\sigma_C$	Reference value of the fatigue strength at 2 million cycles (normal stress)
$\Delta\sigma_D$	Stress range corresponding to the constant amplitude fatigue limit, simply called the "fatigue limit"
$\Delta\sigma_E$	Equivalent stress range of constant amplitude cycles (normal stress)
$\Delta\sigma_L$	Stress range corresponding to the cut-off limit
$\Delta\sigma_R$	Fatigue strength (normal stress)
$\Delta\tau_C$	Reference value of the fatigue strength at 2 million cycles (shear stress)
$\Delta\tau_R$	Fatigue strength (shear stress)
δ_m, δ_1	Initial out-of straightness
ϵ	Strain
η	Coefficient. Coordinate
k_t, k_l	Coefficient
λ	Slenderness
λ_c	Slenderness parameter $= \pi\sqrt{E/f_y}$
$\bar{\lambda}$	Relative slenderness of columns
$\bar{\lambda}_o$	Relative slenderness limit, below which strain hardening effects in columns occur
$\bar{\lambda}_e$	Effective relative slenderness for members with L-sections
$\bar{\lambda}_p$	Relative slenderness of plate
$\bar{\lambda}_i$	Effective relative slenderness for built-up members
μ	Slip coefficient
μ_y, μ_z	Coefficients (flexural buckling)
μ_L	Coefficient (lateral torsional buckling)
ϕ	Lateral torsional buckling coefficient
ψ_y	Cross sectional parameter
ω_y, ω_z	Coefficient (bending moment diagram)

4 DOCUMENTATION OF THE DESIGN

4.1 Calculations

Design calculations shall include

- design assumptions (calculation model),
- action arrangements (including imposed actions),
- material properties,
- properties of connecting devices and
- verification of the relevant limit states.

4.2 Testing

4.2.1 The design may be verified by testing, or by testing combined with calculations.

4.2.2 The magnitude and distribution of actions during tests shall correspond to the design actions for the relevant limit states.

4.2.3 Sample size, scale effects and other relevant effects shall be considered in establishing the design strength of the structure or structural element.

4.3 Documentation

The calculations, drawings, or other relevant documents shall be presented in a manner which is appropriate to the information and documentation requirements.

5 BASIC DESIGN PRINCIPLES

5.1 Objectives and general recommendations

Structures or structural elements shall be designed and maintained such that they, with an appropriate degree of reliability,

- will sustain actions likely to occur

- will perform adequately in normal use

- have a sufficient durability.

These requirements, which can be satisfied by use of this code, shall apply throughout the specified life of a structure, including the period of construction.

The degree of reliability should be chosen to account for the possible consequences of exceeding the design criteria of the limit states. These consequences will vary. The following classification is appropriate:

- risk to life is negligible and economic consequences are small or negligible

- risk to life exists and/or economic consequences are considerable

- risk to life is high and/or economic consequences are great.

The choice of structural concept should also take into account accidental events and their possible consequences. The main structure should as far as practical not be damaged to an extent which is disproportionate to the accidental event.

The design of steel structures should aim at a ductile behaviour, avoiding brittle fracture by appropriate choice of materials, material thickness, connections and selection of details and fabrication methods. See also 6.3.2.3.

5.2 Limit states

The structural performance of a whole structure or parts of it shall be described with reference to limit states.

The limit states are classified into the following two categories, which may also be subclassified:

a) The ultimate limit states.

b) The serviceability limit states.

Ultimate Limit States correspond to:

- overturning of the structure, or parts of the structure;
- rupture of critical sections of the structure due to exceedance of the material strength;
- transformation of the structure into a mechanism (collapse);
- loss of stability (buckling, etc.);
- excessive displacements or deformations, leading to a change of geometry, which necessitates replacing the structure;
- failure of a structure or a member subjected to repetitive actions (fatigue).

The Serviceability Limit States correspond to:

- deformations which affect the normal use or performance of structural or non-structural elements;
- oscillations producing discomfort or affecting structural or non-structural elements or equipment;
- local damage, including limited cracking, which reduces the durability of a structure or affects the performance of structural or non- structural elements.

5.3 Design situations and member resistance

5.3.1 General

All relevant limit states shall be considered in design. A calculation model shall be established for each specific limit state.

5.3.2 Design situations

For any structure it is generally necessary to consider several design situations. Corresponding to each of these, there may be different structural systems, different reliability requirements, different design values, and different environmental conditions. The design situations may include permanent, transient and accidental conditions.

5.3.3 Member resistance

5.3.3.1 For the ultimate limit states, the structure shall be designed for sufficient resistance, i.e. strength and/or stability. At every part of the structure the member resistance shall be larger than or equal to the action effects of the relevant ultimate limit load cases.

5.3.3.2 Variable and repetitive actions shall be considered. At every part of the structure the fatigue strength shall be larger than or equal to the effects of the repetetive actions.

5.3.3.3 For the serviceability limit states the structure shall be designed to eliminate unacceptable levels of vibration, deflections or slip under the effects of the relevant serviceability actions.

6 BASIC VARIABLES

6.l General

The design assumptions shall include the necessary set of basic variables. The normal basic variables are the relevant parameters characterizing:

- actions;
- material properties;
- structural geometry;
- environmental conditions.

Other variables shall also be considered, such as uncertainties of calculation models.

6.2 Actions

6.2.1 General

Actions are characterized as

1. Direct actions:
   One or an assembly of concentrated or distributed forces acting on the structure, such as selfweight, imposed specified actions, wind etc.

2. Indirect actions:
   The result of imposed or constrained deformations in the structure, such as temperature effects, settlements, creep etc.

For characteristic values of the actions, reference is made to the relevant ISO standard or to the appropriate national standards.

According to their occurrence in time and to the variation of their magnitude with time, actions are classified as follows:

- permanent actions,
- variable actions,
- repetitive actions,
- accidental actions,
- temporary or transient actions.

A load case comprises a relevant combination of actions.

According to the way in which the structure responds to an action, one may distinguish between

- static actions, which are acting on the structure without causing any significant oscillations of the structure or parts of the structure.
- dynamic actions, which may cause impact effects or significant oscillations of the structure or parts of the structure.
- repetitive actions, which may cause fatigue.

Dynamic actions, which cause impact effects, may be handled as static by an appropriate increase of the magnitude of its corresponding static effect, except for the cases when such dynamic effects are cyclical or repetitive.

6.2.2 Design value

For a specific limit state the design value, F_d, is the representative action or combination of representative actions, F_k, each multiplied by a partial coefficient, γ_f , i.e.

$$F_d = \sum \gamma_f F_k$$

6.3 Materials

6.3.1 General

All material shall be suitable for its intended use.

6.3.2 Structural steels

6.3.2.1 The steel to be used shall conform to the requirements of ISO 630 and ISO 4951 or to the requirements of the appropriate national standards for structural steels.

6.3.2.2 The dimensions and mass of all steel sections and plates and their dimensional and mass tolerances, shall comply with the relevant ISO or national standards.

6.3.2.3 When elements of the structure may be used at low temperatures, consideration shall be given to notch toughness characteristics of the steel to avoid brittle fracture. This is particularly necessary where thick welded constructions are subjected to tensile stresses.

The selected steel shall be of sufficient toughness and the structure shall be designed with specific attention to minimizing notches and stress concentrations.

A higher notch toughness specification may be required for steel which is to be cold formed and welded.

6.3.2.4 Weldability shall be considered when selecting the appropriate grades of steel. Weldability may be determined on the basis of the carbon equivalent value or other relevant parameters. See also 8.9.1.

When an element is stressed in tension normal to its rolling plane via heavy welds on the surface, lamellar tearing shall be considered. See also 8.9.2.7. The risk of lamellar tearing may be reduced by using steel with specified through thickness ductility in the element concerned.

6.3.3 Connecting devices

6.3.3.1 Connecting devices covered by the design rules of this standard are bolts, studs, rivets and welds.

6.3.3.2 All bolts, nuts and washers, including plated components, shall conform to the relevant ISO standards, or to the appropriate national standards. Bolts used for structures covered by this standard, shall not be of higher grade than 10.9, see ISO 898.

Nuts for preloaded bolts or bolts loaded in tension shall be such that stripping failure will not occur prior to bolt failure.

6.3.3.3 All material for riveting shall conform to the appropriate national standard.

6.3.3.4 All welding consumables shall conform to the appropriate ISO, IIW or national standards.

6.3.4 Testing and inspection of materials

Methods for testing and inspection shall be in accordance with the appropriate ISO or national standards.

6.4 Geometrical parameters

Geometry of the structure shall be uniquely described, i.e. shape, size and arrangement of the structure and its elements. Tolerances shall be included if they are important to the resistance of the structure.

6.5 Design value of resistance

The design value of resistance R_d shall be determined by dividing the characteristic value of resistance R by the appropriate resistance factors γ_r, i.e.

$$R_d = R/\gamma_r$$

See ISO 2394, General principles on reliability for structures.

The resistance factor γ_r comprises uncertainties from:

- the possible systematic deviation and the variability of the material properties (such as f_y, f_u and E) and the geometrical cross-sectional dimensions and the derived cross-sectional properties (such as A, I, W, W_p, i),

- the prediction of member resistance, i.e. the deviation between the actual member resistance and the resistance based on calculations, models or tests.

7 ANALYSIS OF STRUCTURES

7.1 General

Calculation models and basic assumptions for the calculations shall represent the structural response according to the limit state under consideration.

The distribution of internal forces and bending moments shall be determined either by calculations or testing.

In proportioning the structure to meet the various design requirements, the methods of analysis given in this chapter shall be used, as appropriate. The distributions of internal forces and bending moments shall be determined under:

- Ultimate limit state actions, to satisfy strength and overturning requirements;

- Serviceability actions, to satisfy the requirements of serviceability;

- Repetitive actions, for the fatigue safety assessment.

7.2 Structural behaviour

The influence of deformations shall be considered.

The analysis referred to in 7.1 shall, where sway effects are significant, include the sway effects produced by the vertical actions acting on the structure in its displaced configuration. For some types of structures where the vertical actions are small and the structure is relatively stiff, and where the lateral displacement resisting elements are well distributed, the sway effect may be insignificant.

In all cases, the details of members and connections should be consistent with the assumptions made in the design, without adversely affecting any other part of the structure.

Effects such as the distortions of semi-rigid connections and the slip in long slotted holes shall be considered for strength and stability at the ultimate limit states.

7.3 Methods of analysis

7.3.1 General

An ultimate limit state for which the structure will have a ductile mode of failure, may be analyzed by either of the methods given in 7.3.2 - 7.3.4.

7.3.2 Elastic analysis

The forces and moments throughout all or parts of the structure may be determined by an analysis which assumes that individual members behave elastically.

Having determined the forces and moments on the basis of an elastic analysis, the resistance of the structural members may be based either on the first yield criterion in accordance with theory of elasticity or on the strength of the cross section in accordance with Chapter 8.

7.3.3 Elastic-plastic analysis

For each combination of actions the forces and moments throughout all or parts of the structure may be determined by an analysis which considers the non-linear force-deformation relationship of the structural parts.

7.3.4 Plastic analysis

For each combination of actions, the forces and moments throughout all or parts of the structure may be determined by a plastic analysis provided that:

a) The steel material exhibits the stress-strain characteristics necessary to achieve moment redistribution.

b) The relevant widththickness ratios meet the requirements of cross-sections of class I, as given in 8.3.3.

c) The members are prevented from premature lateral buckling in accordance with requirements, for instance as given in A.7.3.4.

d) Web stiffeners are supplied at points of concentrated actions where plastic hinges will form. This requirement need not be met at the location of the last plastic hinge.

e) Except for those splices designed and detailed to behave as hinges, splices occurring within the length of a member shall have an adequate deformability and shall be designed to transmit at least 1.1 times the maximum computed force at the splice location. The design strength need not exceed the full resistance of the member but shall be at least 25 % of that resistance.

Unless specific provisions have been made in both the analysis and design, plastic design shall not be used for structures subjected to alternating plasticity which leads to incremental collapse.

For structures subjected to variable actions of high amplitude which could cause repeated plastic deformations leading to incremental collapse, the design action shall not exceed the shake down resistance.

The resistance of the member is the full strength of the cross-section. The behaviour corresponding to this strength is complete yielding under the given actioneffects whether in tension, compression, shear or bending.

8 ULTIMATE LIMIT STATES

8.1 Member design

8.1.1 General

The resistance of the members should be checked against the internal forces and moments, derived from the structural analysis.

In all cases the resistance of the members shall not be less than the effects of the relevant design actions. The member resistance depends on a criterion either of strength or of overall stability.

The member cross-section should be considered at critical positions along its length for each of, or combination of:

- axial tension
- axial compression
- bending about either axis
- directional or torsional shear.

For structures analysed plastically, where a redistribution of moments is taken into account, special requirements are placed on the material behaviour and structural geometry to ensure that the structure will behave as assumed in design, see 7.3.4 and 8.3.3.

8.1.2 Cross-sectional resistance

Cross-sectional strength is determined on the basis of a classification system, see 8.3. The particular classification will depend on the behaviour of the cross-section and on its orientation.

For class 1 and class 2 sections the resistance of a cross section is the full plastic strength. For class 3 and class 4 sections local buckling may occur before the full plastic strength can be attained.

8.1.3 Member stability

Where member instability occurs at the ultimate limit state, the resistance of the member is based on 8.4 and 8.5 for class 1, 2 and 3 sections, and on 8.6 for class 4 sections.

8.2 Resistance of members

8.2.1 Member strength

For class 1 or 2 sections where no instability occurs prior to the ultimate limit state, the resistance of a structural element is based on the full strength of the cross-section. This strength involves yielding under the given actions whether in tension, compression or bending.

The resistance of tension members is governed by yielding or fracture as given in A.8.2.1.1.

For members or parts of members subjected to compression, with the exception of class 4 sections, limitations are imposed on the width-thickness ratios to ensure that premature local instability will not occur, see 8.3.3. For members subjected to bending the section class also depends on the axis about which the members is bent.

For members with a cross-section of class 4 the effects of local buckling shall be adequately included.

8.3 Classification of cross sections

8.3.1 General

Members are designated as class 1, 2, 3 or 4 depending on the maximum width-thickness ratios of their cross-sectional elements subject to compression, and/or bending, and thus on the capability of the member to resist local buckling.

8.3.2 Definitions of classes

Class I: Cross-sections which can develop yielding in the entire cross-section, and permit sufficient rotations to allow redistribution of moments in the structure.

Class 2: Cross-sections which can develop yielding in the entire cross-section, but local buckling prevents sufficient rotations at constant moment and therefore limits redistribution of moments in the structure.

Class 3: Cross-sections which can attain the yield strain at the extreme fibres of the compression zone, and, because of local buckling, are prevented from developing full plasticity.

Class 4: Cross-sections which do not qualify as class 1, 2 or 3 sections, and, because of local buckling, are prevented from attaining gross yielding in compression.

Class I cross-sections shall, when containing a plastic hinge, have an axis of symmetry in the plane of the action. Class 1 cross-sections without an axis of symmetry may be used when the design does not require plastic hinges, in which case lateral bending and torsion must be taken into account.

Class 2 cross-sections shall, when subjected to flexure, have an axis of symmetry in the plane of the action, unless the effects of possible asymmetry are included in the analysis. By use of a reduced yield stress f_{ye}, see A.8.3.2, or by using a width reduced to meet Class 1 section requirements, such sections may be regarded as class 1 sections.

Class 2 cross sections may be used for plastic design provided that the rotation capacity required at locations of plastic hinges can be achieved by the section.

Local buckling does not further govern the design of a member if the proportions of the cross-section do not exceed the limiting values given in Table A.8.3.3 for class I and 2 cross-sections.

8.3.3 Maximum width-thickness ratios of elements subjected to compression and/or bending

Maximum width thickness ratios of cross-sectional elements subjected to compression and/or bending shall be given in the national standards.

Recommended width-thickness ratios for cross-sections of class 1, 2 and 3 are given in Table A.8.3.3. For class 4 sections the member resistance may be determined by the methods given in A.8.6.

8.4 Flexural buckling

8.4.1 Effective buckling length

The effective length L_E of a compression member may be defined as $L_E = K_E L$, where

$$K_E = \sqrt{\frac{N_E}{N_{Ecr}}} \; ; \qquad N_E = \frac{\pi^2 EI}{L^2}$$

and N_{Ecr} is the theoretical elastic buckling load of the actual column, computed with due regard to rotational and translational restraints, and L is the geometrical length of the member, i.e. distance between the centres of joints.

8.4.1.1 For compression members in trusses or frames the effective length should be considered for buckling both in and out of the plane of the truss or frame.

8.4.1.2 The effective in-plane buckling length of compressed members in trusses or frames should be estimated with due consideration of the cross-section of the members and their end restraints. When estimating out of plane effective lengths of compressed members, the end restraint of the members and the resistance to displacements of the restraining members out of the plane of the trestle frame should be taken into consideration.

8.4.1.3 For structures with moment resisting frames in which the sway effects have been included in the analysis to determine the moments and forces in the members, or for structures in which the sway effects, in addition to the lateral actions, are resisted by bracing or shear walls, the effective length factor, K_E, shall be taken equal to 1.0, unless the degree of rotational restraints afforded at the ends of the unbraced lengths show that a value of K_L less than 1.0 is applicable.

For structures with moment resisting frames in which sway effects have not been included in the analysis used to determine the design moments and forces, the effective length factor shall be determined from the degree of rotational and translational restraints afforded at the ends of the unbraced length, but shall not be less than 1.0.

8.4.2 Slenderness

The relative slenderness is defined by

$$\bar{\lambda} = \sqrt{\frac{N_r}{N_{Ecr}}}$$

where N_r is the cross-sectional normal force resistance and N_{Ecr} is the elestic buckling load.

The relative slenderness for class 1, 2 and 3 sections is the slenderness λ divided by a factor λ_c

$$\bar{\lambda} = \frac{\lambda}{\lambda_c} \quad \text{where} \quad \lambda_c = \pi \frac{\sqrt{E}}{f_y}$$

The relative slenderness is introduced to obtain formulae and diagrams which are independent of the yield strength of the steel material.

The slenderness λ should be taken as the effective length L_E divided by the radius of gyration of the cross-section, calculated with respect to its relevant axis of buckling:

$$\lambda = \frac{L_e}{i}$$

Due to serviceability and erection requirements, limitations for the slenderness of compression members may be given in national standards. See also 9.2 regarding vibration considerations.

8.4.3 Compression resistance

Compression resistance of a member subjected to an axial force depends on the slenderness g, the yield strength f_y and the cross-sectional properties.

For cross-sections of class I, 2 or 3, see 8.3.3, the resistance is

$$N_{cd} = \frac{1}{\gamma_r} f_c \cdot A_g = f_{cd} \cdot A_g$$

For a column with a cross-section of Class 4 the compression resistance must also include the effects of local buckling.

8.4.4 Buckling strength f_c

Buckling curves for the determination of f_c should represent the maximum load resistance of the actual member. When using the curves a computational model with effective length L_E and simple end conditions, i.e. translational restraints only, is adopted.

The following shall be accounted for in the derivation of the buckling curves:

a) magnitude and distribution of the residual stresses
b) out-of-straightness due to manufacture

The actual curve to be used will depend on the cross-sectional shape, manufacturing process as it affects residual stresses and out-of-straightness, and the axis of buckling.

Buckling curves given in A.8.4.4, National Standards, or the Structural Stability Research Council's Guide to Stability Design Criteria for Metal Structures, accounting for the preceding effects, are considered acceptable.

8.4.5 Compression members subject to moments

The resistance of compression members subject to moments may be checked using interaction equations as given in national standards. Such interaction equations shall

a) provide sufficiently good statistical correlation with experimental results or numerical simulations so that resistance factors can be evaluated.

b) take into account such effects as the amplification of bending moments due to the axial force acting on the deformed shape of the member, the variation of bending moments along the length of the member, torsional or lateral torsional buckling; initial out-of-straightness, residual stresses.

c) give valid results for any combinations of N, M_x and M_y.

Examples of interaction equations are given in A.8.4.5.

8.4.6 Buckling of built-up members

Built-up members shall be calculated considering the possibility of local buckling of the different parts of the member. The strength and the stiffness of the different parts of the member shall be such that the assumed buckling mode may be attained.

Buckling of built-up members may be calculated as given in 8.4.3 - 8.4.5. The slenderness of such a member shall be increased due to shear deformations. See A.8.4.6.

Built up members where the different parts of the cross-section is continuously connected to the rest of the section, shall be calculated considering the entire cross section as one part.

8.5 Torsional and lateral torsional buckling

8.5.1 Torsional buckling

The torsional buckling resistance of a column with a cross-section of class 1, 2 or 3 may be expressed as

$$N_{Td} = \frac{1}{\gamma_r} f_{cT} \cdot A_g$$

where f_{cT} is the torsional buckling strength.

For a column with a cross-section of class 4, the torsional buckling resistance shall also include the effects of local buckling.

8.5.2 Lateral torsional buckling

The lateral torsional buckling resistance of a beam may be expressed by

$$M_{Ld} = \frac{1}{\gamma_r} f_{cL} \cdot W_p$$

for class 1 and 2 sections

$$M_{Ld} = \frac{1}{\gamma_r} f_{cL} \cdot W$$

for class 3 sections, and by

$$M_{Ld} = \frac{1}{\gamma_r} f_{cL} \cdot W_e$$

for class 4 sections, unless a more detailed analysis is made, where f_{cL} is the lateral torsional buckling strength.

Members in structures where the distribution of moments and forces have been determined by plastic analysis shall be braced to resist lateral and torsional displacements at, or immediately adjacent to, all hinge locations. (See e.g. A.7.3.4)

Calculation of elastic lateral torsional buckling strength should for Class 4 sections be based on the net effective section.

8.5.3 Buckling strengths f_{cT} and f_{cL}

The buckling strengths f_{cT} and f_{cL} for torsional and lateral torsional buckling, respectively, shall include all relevant effects and provide sufficient correlation with experiments and/or numerical simulations.

The buckling strengths f_{cT} and f_{cL} shall be given in the national standards, or as recommended in A.8.5.3.

8.5.4 Bracing of beams, girders and trusses

8.5.4.1 Bracing members assumed to provide lateral support to the compression flange of beams and girders, or to the compression chord of trusses, and the connections of such bracing members, shall be proportioned to resist a force equal to 1 per cent of the force in the compression flange or chord at the point of support, unless more accurate calculations are made.

8.5.4.2 The stiffness of bracing members shall be sufficient to restrict the increase of out-of-plane deflections at the braced point to a value equal to or close to the initial out-of-plane imperfection of the braced member.

8.5.4.3 When bracing of the compression flange or chord is effected by a slab or deck, the slab or deck, and the means by which the computed bracing forces are transmitted between the flange or chord and the slab or deck, shall be adequate to resist a force in the plane of the slab or deck.

8.5.4.4 The stiffness of the slab or deck shall be sufficient to enable the braced member ro reach its design resistance. In assessing the stiffness, the structural stiffness of the deck or slab, the flexibility of the connecting elements and the flexibility of the anchoring portion of the structure, shall be considered.

8.5.4.5 Consideration shall be given to the probable accumulation of forces from one braced member to another.

8.5.4.6 Members restraining beams and girders designed to resist actions causing torsion shall be proportioned to resist these effects. Special consideration shall be given to the connections of asymmetric sections such as channels, angles and zees.

8.6 Buckling of plates

8.6.1 General

The calculation of the ultimate resistance of a plate may be based on a large deflection theory or on test results. At its ultimate resistance the plate has buckled and the in-plane stresses are concentrated at the supported plate edges. In these regions stresses may reach the yield stress.

For design calculations the concept of an effective area may be used.

The effective area is a reduced area which, multicplied by the yield stress, will give the same ultimate resistance as obtained from tests on the entire plate.

8.6.2 Uniaxial force or in-plane moment

8.6.2.1 The resistance of a plate subject to uniaxial force or in-plane moment may be expressed as a function of f_y and of the relative slenderness λ_p, defined by

$$\bar{\lambda}_p = \frac{f_y}{\sigma_{cr}} = 1{,}05\ \frac{b}{t}\ \frac{1}{\sqrt{k}} = \sqrt{\frac{f_y}{E}}$$

where the coefficient k depends on the stress distribution and the support conditions. Normally, the edge should be assumed to be simply supported or free, unless it can be proved that the edge is effectively restrained or fixed.

8.6.2.2 The moment resistance of a web may be calculated according to A.8.6.2.2. If the effective flanges of an I-beam subjected to bending moment can sustain the entire moment alone, the web may be exclusively retained for the purpose of carrying an additional shear force. Otherwise, the combined buckling effects of shear and bending shall be considered.

8.6.2.3 The width-thickness-ratio of a web should be limited such that the compression flange is prevented from buckling into the web.

8.6.2.4 For web panels with varying bending moment and axial force the design stress distribution may be calculated according to A.8.6.2.4.

8.6.3 Shear resistance of webs

8.6.3.1 The shear resistance of webs may be calculated according to the method given in 8.6.3.2. Alternatively, the shear resistance of webs may be calculated according to a tension field theory, which has been sufficiently correlated to experimental results and/or numerical simulations.

8.6.3.2 The shear resistance of webs may be expressed as a function of f_y and the relative slenderness λ_p, defined by

$$\bar{\lambda}_p = \sqrt{\frac{\tau_y}{\tau_{cr}}} = 0.8 \cdot \frac{h}{t_w} \cdot \frac{1}{\sqrt{k_s}} \cdot \sqrt{\frac{f_y}{E}}$$

where $\tau_y = f_y/\sqrt{3}$ and k_s is the elastic shear buckling coefficient depending on the aspect ratio, see A.8.6.3.2.

The resistance of a web also depends on the flexural rigidity of the end stiffeners in the web plane.

The resistance is given by

$$V_{cd} = \frac{1}{\gamma_r} \cdot \tau_c \cdot t_w \cdot h$$

Recommended values for the buckling strength τ_c is given in A.8.6.3.2.

The shear resistance should be checked against the largest shear force within the panel.

A rigid end stiffener shall satisfy the requirements in 8.6.5.4 and 8.6.5.5.

8.6.4 Combined forces

Combined forces may be handled by simplified interaction formulae. Recommended formulae, based on 8.6.2 and 8.6.3.2, are given in A.8.6.4.

8.6.5 Webs or panels subdivided by stiffeners

8.6.5.1 The purpose of stiffeners is to subdivide a plate or a web into smaller panels to increase the resistance. The smaller panels shall be calculated as given in 8.6.2 to 8.6.4, see Fig. 8.6.5. Stiffeners shall be designed such that they provide sufficient stiffness and strength to allow the required resistance of the plate or web to be developed.

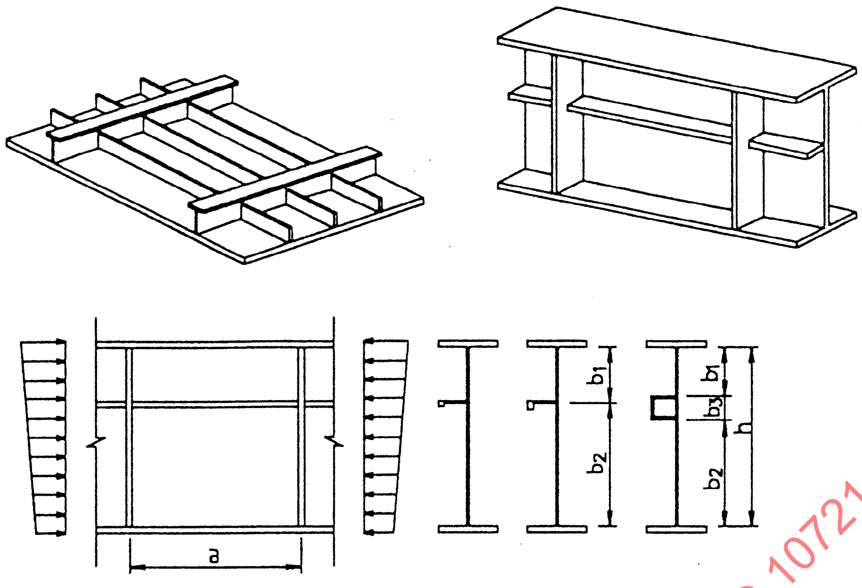


Fig. 8.6.5 Examples of transverse and longitudinal stiffeners

Unless more accurate calculations are made, the stiffeners shall comply with the requirements given in 8.6.5.2 - 8.6.5.5.

For structures subject to repetitive actions the unfavourable fatigue effects of possible non-continuous longitudinal stiffeners, or terminated transverse and support stiffeners, shall be considered.

8.6.5.2 A transverse stiffener shall have sufficient strength to carry all the forces which are transferred to it, and have sufficient stiffness to allow the required resistance of the adjoining panels to be developed.

Recommended requirements for the second moment of area and the cross section area of transverse stiffeness are given in A.8.6.5.2.

8.6.5.3 A longitudinal stiffener shall have sufficient strength to carry all the forces which are transferred to it, and have sufficient stiffness to allow the required resistance of the adjoining panels to be developed.

Recommended requirements for the second moment of area of longitudinal stiffeners intended to form a rigid support for adjacent panels are given in A.8.6.5.3.

8.6.5.4 Stiffeners at supports should be checked for the total support force. The buckling length is equal to 0.75 h, where h is the web plate height or the distance between flanges. Parts of the web may be considered as acting together with the stiffener.

The stiffener may be stopped at a distance not less than 4t and not larger than 6t from the unloaded flange. This also applies to intermediate transverse stiffeners.

8.6.5.5 A stiffener at the end of a web shall be able to carry all the forces which are transferred to it from the adjoining web, see A.8.6.5.5.

8.7 Connections, general requirements

Connections may be designed to transmit forces through fasteners of different types.

The design shall be based on the strength of the individual connectors or welds.

All connections shall have a design strength such that the structure is capable of resisting the design forces.

The structural properties of connections shall be such that the assumptions made in the analysis and design of the structure are achieved. Connections may be classified according to their rigidity and/or strength.

Connections may be designed by distributing the internal forces in a rational manner provided that they are in equilibrium with the applied design forces, that the design resistance is nowhere exceeded and that the adopted distribution does not entail excessive deformations.

Should there be significant deformations, their structural effects shall be considered.

If the design strength of the connection is less than that of the connected members, the deformation capacity of the connection shall be sufficient for the assumed failure mode to be reached. This deformation capacity shall be demonstrated by experiments or calculations.

When various types of fasteners are used to carry a shear force in the same plane or when welding and fasteners are combined, then one type of connection should normally be designed to carry the total action. Welds and preloaded high-strength bolts in slip-critical connections may, however, be assumed to share the forces at the serviceability load level provided the bolts are fully tightened after welding, see 8.8.3.2.b.

8.8 Bolted connections

8.8.1 General

The fasteners used in bolted connections may either be ordinary bolts or high-strength bolts, as described below. High-strength bolts may be non-preloaded or preloaded, as described in 8.8.2.3.

8.8.1.1 Ordinary bolts are those which are manufactured from low-carbon steel. The steel used shall conform to the requirements of ISO 4753 or to the requirements of the appropriate national standards.

8.8.1.2 High strength bolts are those which are manufactured of high-strength steel. The steel used shall conform to the requirements of the appropriate ISO or national standards.

8.8.1.3 Non-preloaded bolts may be used as fastening elements in connections with shear, tension, or combined shear and tension in the bolts. They should not be used in joints which are subjected to fatigue or earthquake actions and they should not be used in structures sensitive to geometric changes, as joint slip may occur. See also 8.8.2.4.

8.8.1.4 High-strength bolts installed in accordance with the provision of 8.8.2.3 have a closely controlled preload in the bolts. They may be used as the fastening elements in connections which produce shear, tension, or combined shear and tension, or in slip-critical connections.

8.8.2 Bolting details

8.8.2.1 Spacing of bolts should comply with the requirements in A.8.8.2.1.

8.8.2.2 The nominal diameter of the bolt hole shall not be more than 2 mm larger than the nominal bolt diameter for bolts smaller than 27 mm and 3 mm for bolts equal to 27 mm diameter and larger. Oversize or slotted holes may be used for 16 mm or larger bolts.

Joints using oversize or slotted holes shall meet the requirements given in A.8.8.2.2 or as given in the national standard.

Oversize holes shall not be used in bearing-type connections but may be used in any or all plies of slip-critical connections.

8.8.2.3 Each preloaded high-strength bolt shall be tightened to provide, when all bolts in the joint are tight, a tensile force equal to or greater than 70 % of the nominal ultimate tensile strength of the bolt.

8.8.2.4 Ordinary bolts and non-preloaded high-strength bolts with normal clearance between hole diameter and shaft diameter shall not be used in connections where the force direction is frequently changing.

8.8.3 Strength of connections with bolts and rivets

8.8.3.1 For calculation of bolts in tension, see A.8.8.3.1. It is important that possible prying forces are included. Such connections shall not be used with non-preloaded bolts if the connection is subject to frequent variation in tension, see also chapter 10.

8.8.3.2 Bolted joints subjected to shear forces may be designed according to the following

a) Bearing type connections with normal bolts or high strength bolts with no controlled tightening up to grade 10.9:
The shear force at the ultimate limit state should not exceed the bearing resistance nor the shear resistance as given in National Standards or in A.8.8.3.2.1.

b) Slip-critical connections with preloaded high strength bolts where slip shall not occur at the serviceability limit state:
The shear force at the serviceability limit state should not exceed the slip resistance as given in National Standards or in A.8.8.3.2.2.
The shear force at the ultimate limit state should not exceed the bearing resistance nor the shear resistance as given in National Standards or in A.8.8.3.2.1.

8.8.4 Slip coefficients

The slip coefficient μ depends on the conditions of the faying surfaces of the parts. Representative values of the slip coefficient are given in Table A.8.8.4.

8.8.5 Deduction for holes

8.8.5.1 In deducting holes for fasteners, the hole diameter should be used, not the diameter of the fastener.

For countersunk holes, the area to be deducted should be the gross area for the hole, including the countersunk portion, in plane of its axis.

8.8.5.2 When holes are not staggered the area to be deducted from the gross sectional area should be the maximum sum of the section area of the holes in any cross section, at right angles to the direction of stress in the member

8.8.5.3 When holes are staggered the area to be deducted should be the greater of:

a) Deduction for non-staggered holes.

b) The sum of the sectional areas of all holes in any zig-zag line extendig progressively across the member or part of the member, minus $s^2t/4g$ for each gauge space in the chain of holes.

s = the staggered pitch, i.e. the distance, measured parallel to the direction of stress in the member, centre-to-centre of holes in consecutive lines.

t = the thickness of the plate

g = the gauge, i.e. the distance, measured perpendicular to the direction of stress in the member, centre-to-centre of holes in consecutive lines.

For sections such as angles with holes in both legs, the gauge should be measured along the centre of thickness of the plate.

In a built-up member, where the chain of holes, considered in individual parts, do not correspond with the critical chain of holes for the member as a whole, the resistance of any fasteners joining the parts between such chains of holes should be taken into account in determining the resistance of the member.

8.8.5.4 Rivets and fitted bolts shall be calculated as regular non-preloaded bolts. When making alterations, rivets and new high-strength bolts in slip-critical joints may be considered as sharing forces due to selfweight and imposed specified actions.

Rivets shall when possible not be used for connections giving tensile forces in the rivets.

8.8.6 Length of connection

For a distance less than 15·d between the first and the last bolt in a connection, the resistance of the joint can be taken as the sum of the resistances of the individual bolts. d is the diameter of the bolt. When the distance is larger than 15·d, the resistance of the joint shall be reduced, see A.8.8.6.

However, this reduction does not apply for uniform distribution of forces over the connection length, e.g. for the transfer of shear forces from the web of a beam or column to the flange, neither does it apply to slip critical connections.

8.9 Welded connections

8.9.1 Scope

The provisions of this section are intended to apply to:

1) welded joints in steel structures subjected primarily to static actions. For provisions applicable to steel structures under fatigue actions, see chapter 10.

2) weldable structural steels meeting the requirements of 6.3.2. Suitability for welding shall be established by reference to the relevant national welding standard. In particular the Carbon Equivalent Value (calculated for instance in accordance with the IIW formula) may need to be controlled, and also the sulphur level may need to be controlled in heavy welded joints carrying through thickness tensile stresses.

3) material thicknesses of 4 mm and larger.

4) welding with arc welding processes

5) joints in which the weld metal is compatible with the base metal in terms of mechanical properties, as defined in the relevant national standards.

8.9.2 General requirements

8.9.2.1 Welded structures shall be designed to permit adequate access for welding and inspection during construction.

8.9.2.2 Complete information regarding location, type, size, and length of all welds shall be shown on the drawings. The drawings shall distinguish between shop and field welds.

8.9.2.3 In the case of partial joint penetration grooves, detailed shop or working drawings shall specify the groove depths applicable to the effective throat required.

8.9.2.4 Where it is important to minimize shrinkage stresses and distortions, special fabrication procedures shall be established.

8.9.2.5 The required weld lengths specified on the drawings shall be the effective lengths.

8.9.2.6 Inspection requirements shall be defined on the plans or in the specifications.

8.9.2.7 Where tensile stresses occur via heavy welds perpendicular to the surface of the element, attention should be given to the susceptibility of the steel material to lamellar tearing. If such details cannot be avoided, appropriate measures shall be taken to minimize the possibility of lamellar tearing. For flat elements with a thickness of more than 16 mm the combination of welding procedure and/or the through-thickness properties of the material and/or the joint detail (see e.g. Fig. 8.9.2.7) shall be such as to avoid lamellar tearing.

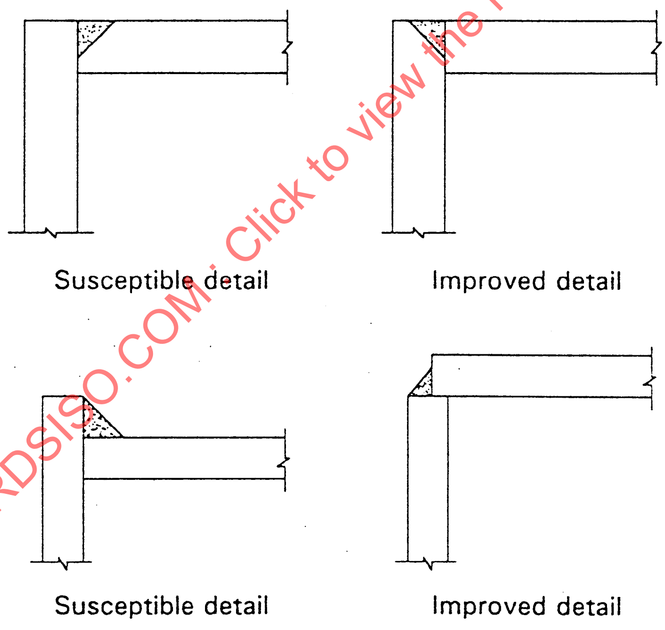


Fig. 8.9.2.7

8.9.2.8 Eccentricity between intersecting parts and members shall be avoided unless accounted for in the design.

If it cannot be avoided, adequate design provisons shall be made for all eccentricities. In the cases of end connections, of single angles, double angles and similar type of members, it is not necessary to completely balance the welds about the neutral axis or axes of such members.

8.9.2.9 Welds at the location of plastic hinges shall be able to develop the full resistance of the connected parts.

8.9.2.10 For ordinary structural steels welding in cold formed areas is allowed, provided appropriate measures are taken to avoid the possibility of brittle fracture.

8.9.3 Types of welds

8.9.3.1 For the purpose of this Standard, welds shall be generally classified as groove, fillet, plug or slot welds.

8.9.3.2 A complete penetration groove weld is defined as one having complete penetration and fusion of weld and base metal throughout the depth of the joint, see 8.9.5.7.

8.9.3.3 A partial penetration groove weld is defined as one having weld penetration less than the full thickness of the joint, see 8.9.5.8 and 8.9.5.9.

8.9.3.4 National standards shall be consulted for additional conditions defining complete and partial penetration groove welds.

8.9.3.5 Flare grooves shall be classified as partial penetration groove welds, see also 8.9.7.11.

8.9.3.6 Groove welds shall be continuous for the full length of the joint, except as provided in 8.9.3.7 or as otherwise permitted.

8.9.3.7 Members of an assembly connected by groove or fillet welds throughout their length may, at points of external framing, have additional welds to accommodate the external action, but such welds need not be continuous for the full length of the members.

8.9.3.8 Fillet welds may be continuous or intermittent.

Except as permitted in national standards, fillet welds shall not terminate at corners of parts or members, but shall be returned continuously, full sized, around the corner for a length equal to twice the weld size where such return can be made in the same plane. End returns shall be indicated on drawings.

8.9.3.9 All corners of slots provided for fillet welding shall be rounded and the fillet welds shall extend completely around the periphery of the slots.

8.9.3.10 Groove and fillet welds may be used to transmit any combination of forces.

Single fillet and single partial penetration groove welds shall not be subjected to bending about the longitudinal axis of the weld if it produces tension at the root of the weld.

Fillet welds may be used for connecting parts, of which the fusion faces form an angle of 60° to 120°, see Fig. 8.9.7.2. Angles of less than 60° are permitted. However, in such cases the weld shall be calculated as a partial penetration groove weld. For angles larger than 120° the fillet welds shall not be relied upon to transmit calculated forces.

Plug and slot welds in lap joints may be used to transmit shear or to prevent buckling or separation of lapped parts.

8.9.4 Design assumptions

8.9.4.1 For welded joints or single welds, designed to transfer any combination of loads, the forces within the joints or the welds at their line of action shall be established by either an elastic or a plastic analysis.

8.9.4.2 Although the distribution of stresses along the length of the weld may be uneven, such distributions can, in most cases, be considered uniform. However, other stress distributions may be assumed provided they satisfy the basic requirements of equilibrium and continuity and they adequately relate to the actual deformation characteristics of the joint.

8.9.4.3 Residual stresses and stresses not participating in the transfer of forces need not be considered in the design of welds subjected to static actions. This applies specifically to the normal stress parallel to the axis of the weld which is accommodated by the base material.

8.9.5 Design provisions

8.9.5.1 The strength of base metals shall be those specified in the applicable national standard.

The strength of the weld metal shall normally be equal to or greater than the strength of the weakest base metal.

8.9.5.2 The resistance of welded joints shall be checked as recommended in A.8.9.5, A.8.9.6 and A.8.9.7 or as specified in the applicable national standard.

8.9.5.3 The design value for shear in the base metal is the shear strength, which, unless otherwise specified in the applicable national standard, is taken as the limiting value of F given in A.8.9.7.1.

8.9.5.4 The vector sum of longitudinal and transverse shear forces shall not exceed the strength requirements given in A.8.9.7.1, unless an alternative acceptable ultimate strength analysis is used.

8.9.5.5 Plug and slot welds shall be considered only to provide shear resistance in the plane of the connected parts.

8.9.5.6 The effective area of groove welds shall be the effective weld length multiplied by the effective throat thickness.

The effective weld length for any groove weld, square or skewed to the direction of stress, shall be the width of the parts which are joined.

8.9.5.7 The effective throat thickness of a complete penetration groove weld shall be the thickness of the thinner part joined, and no increase is permitted for weld reinforcement.

The effective throat thickness of a partial penetration groove weld shall be as defined in the relevant national standard.

The compressive resistance of joints utilizing partial penetration groove welds shall be based on the effective throat area of the welds plus the area of the base metal fitted in contact bearing.

8.9.5.8 The effective throat thickness of a partial penetration groove weld for joints with no root opening shall be the depth of chamfer, minus 2 mm for grooves having an included angle at the root of the groove less than 60° but not less than 45°.

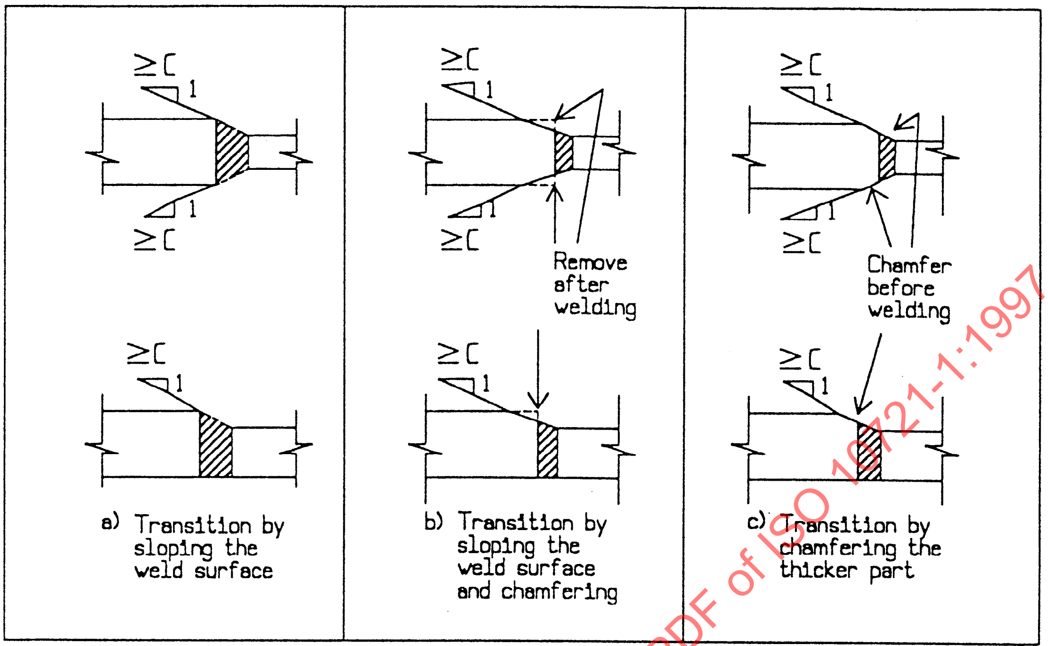
The effective throat thickness of a partial penetration groove weld shall be the depth of chamfer for grooves having an included angle at the root of the groove of 60° or greater.

8.9.5.9 The effective throat thickness of a partial penetration groove weld reinforced with a fillet weld shall be the shortest distance between the root of the groove and the surface of the fillet, minus 2 mm where such reduction is required by 8.9.5.8.

8.9.6 Complete joint penetration groove welds in butt and tee joints

8.9.6.1 Tension butt joints in plates of different material thicknesses or widths shall be made in such a manner that the slope through the transition zone is not steeper than 1 in 1, except for structures subject to fatigue, in which case the slope should not be steeper than 1 in 4. The transition shall be accomplished by chamfering the thicker part, tapering the wider part, sloping the weld metal, or by any combination of these, see Fig. 8.9.6.1 a and b.

Compression butt joints do not require a transition zone in members of different thickness or width.



C = 4 for structures subject to fatigue
C = 1 for structures not subjected to fatigue

Fig. 8.9.6.1.a Transition of butt joints in parts of unequal thickness

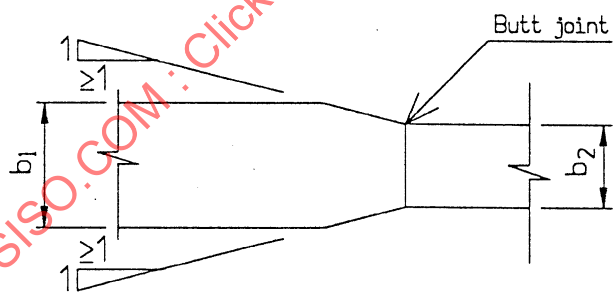


Fig. 8.9.6.1.b Transition of width

8.9.6.2 Partial penetration groove welds are permitted for steels with good ductility in butt, tee and corner joints. When required such joints may be reinforced with fillet welds, see Fig. 8.9.6.2.

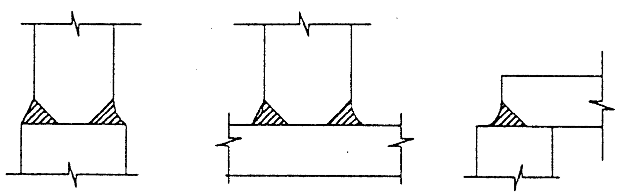


Fig. 8.9.6.2 Partial penetration groove welds

8.9.7 Fillet welds

8.9.7.1 A uniform stress distribution may be assumed along the length and over the throat section of fillet welds.

8.9.7.2 The effective cross-sectional area of a fillet weld shall be the area of the largest triangle which can be fully inscribed within the fusion faces and the weld surface, provided there is a minimum root penetration, but with such penetration not taken into account. The throat thickness, or the "a" dimension, shall be the height of the largest inscribed triangle, see Fig. 8.9.7.2.

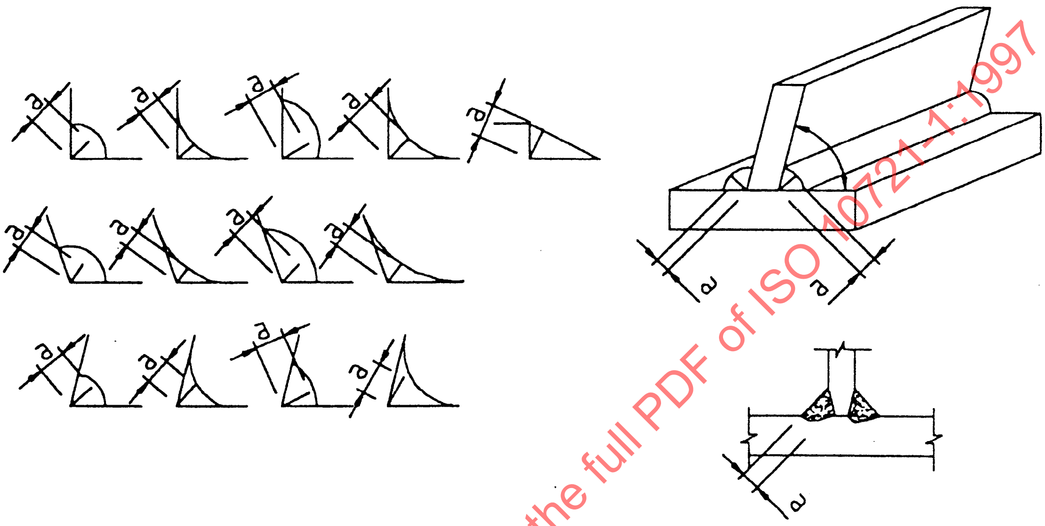


Fig. 8.9.7.2 Throat thicknesses for fillet welds

8.9.7.3 For automatic submerged arc welding and for welding with deep penetration electrodes, which are recognized as such by national standards, the throat dimension "a" may be increased by 20%, but not by more than 3 mm.

8.9.7.4 Unless otherwise specified in national standards the effective length of a fillet weld shall be the overall length of the full-size fillet, including end returns. No reduction in effective length shall be made for neither the start nor the termination of the weld if the weld is full size throughout its length. The minimum effective fillet length shall be a length equal to six times the throat of the weld (6a) or 40 mm, whichever is larger.

8.9.7.5 The effective weld length of a fillet weld shall be used for strength calculations and be shown on design drawings. The effective weld length shall be the required length to safely transfer the design forces. There will be no upper limit for the length of the weld when the stress distribution along the weld corresponds to the stress distribution in the adjacent base metal.

8.9.7.6 The minimum throat thickness is 3 mm. The throat thickness shall not be larger than that required to balance the strength of the adjacent base metal.

8.9.7.7 Fillet welds shall preferably be made with equal leg sizes and with reasonably flat faces.

8.9.7.8 Intermittent fillet welds may be used to carry calculated forces.

8.9.7.9 Fillet welds in holes or slots may be used to transmit shear, to prevent buckling or for separation of joined parts. Fillets welds in holes or slots are not to be considered as plug or slot welds. The minimum diameter of holes or width of slots shall not be less than the thickness of the part containing it plus 8 mm. The ends of slots shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except for those ends which extend to the edge of the part.

8.9.7.10 The leg size of fillet welds reinforcing groove welds, for smoother transition in T- and corner joints, shall not be less than t/4 where t is the thickness of the welded member, but need not be more than 10 mm. Such reinforcement is mandatory for T-joints subjected to fatigue actions, Fig. 8.9.7.10. See also chapter 10, Fatigue.

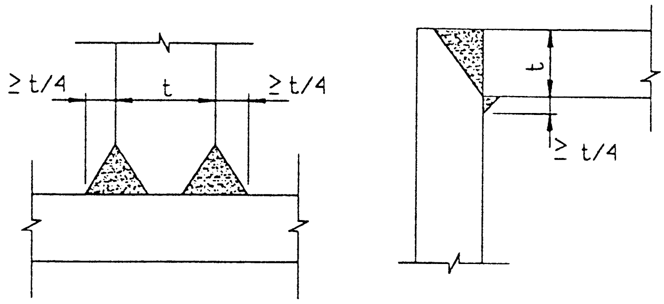


Fig. 8.9.7.10 Reinforcement for T- and corner joints

8.9.7.11 The effective throat thickness

- of flare-V- and flare-bevel-groove welds in joints of hollow rectangular sections, see Fig. 8.9.7.11.a, and

- of flare-groove welds for solid bars fitted flush to the surface of the solid section of the bar, see Fig. 8.9.7.11.b,

shall be as defined in the relevant national standard.

The effective throat thickness may be established by means of trial welds for each set of procedural conditions, and the trial welds should be sectioned and measured to obtain welding techniques that will ensure that the design throat thickness is achieved in production.

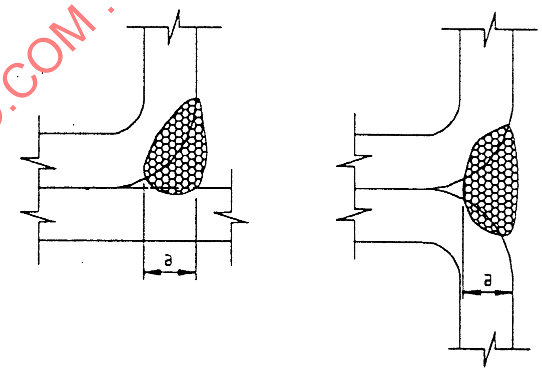


Fig. 8.9.7.11.a Flare grooves in RHS joints

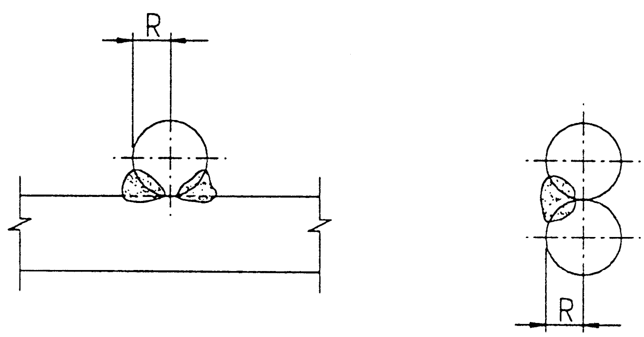


Fig. 8.9.7.11.b Flare grooves in solid section joints

8.9.8 Plug and slot welds

8.9.8.1 The diameter of the hole for a plug weld or the width of a slot for a slot weld shall not be less than the thickness of the part containing it plus 8 mm.

8.9.8.2 The ends of the slot shall be semi-circular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except for those ends which extend to the edge of the part.

8.9.8.3 The thickness of plug or slot welds in plates of 16 mm thickness or less shall be equal to the thickness of the material.

The thickness of plug or slot welds in plates thicker than 16 mm shall be at least one-half of the thickness of the plate, but not less than 16 mm.

8.10 Joints in contact bearing

8.10.1 Compression forces between different parts in welded joints may be transmitted by contact bearing provided that the contact surfaces are parallel and fit reasonably well. Small local lack of fit due to irregularities of the mating surfaces of up to 2 mm is permitted. Contact surfaces shall be locked against lifting or sliding, unless otherwise specified or in agreement with accepted practice.

For structures subject to fatigue, see chapter 10.

8.10.2 Provisions shall be made to prevent lateral displacement. Friction forces may be taken into account. Tensile and shear forces and forces which may be generated by buckling shall be carried by joints designed for such forces.

9 SERVICEABILITY LIMIT STATES

9.1 The evaluation of the serviceability limit states shall be based on the representative actions.

9.2 Serviceability limit states shall comprise considerations of

- Deformations which affect the efficient use or the appearance of structural or non-structural elements.

- Vibrations producing discomfort or adversely affecting structural or non-structural elements and equipment, especially if resonance occurs. Resonant oscillations of slender members exposed to wind action or water flow shall be considered.

- Local damage, including cracking, which reduces the durability of a structure or affects the efficiency or appearance of strucutral or nonstructural elements.

- Durability, the structure shall withstand the actions for the expected lifetime of the structure, or plans for maintenance work shall be given.

- Any other criteria arising from special functional requirements.

9.3 In the serviceability limit states normally two types of combinations are appropriate:

- Combination for short term action effects, such as combinations of permanent actions and variable actions with frequent values.

- Combination for long-term action effects, such as combinations of permanent actions and variable actions with quasi-permanent (sustained) values.

9.4 For the serviceability limit states, the calculations shall normally be carried out in the elastic field. However, a restricted plastic redistribution of forces and moments can be accepted, provided the plastic redistribution is not repeated, and it is included in the calculation of deformations.

10 FATIGUE

10.1 Scope

10.1.1 General

These rules present a general method for the fatigue assessment of structures and structural elements which are subjected to repeated fluctuations of stresses.

The fatigue assessment procedures assume that the structure has been designed in accordance with the other limit state requirements of this standard and the material conforms to the properties required in 6.3.

10.1.2 Limitations

The rules are applicable for all structural steel grades in accordance with this standard. Bolts are acceptable up to ISO grade 10.9 or equivalent.

All nominal stresses for fatigue assessment must be within the elastic limits of the material, the range of such stresses (unfactored) should not exceed 1.5 f_y for normal stresses and 1.5 $f_y/\sqrt{3}$ for shear stresses.

The following effects are not covered by the rules of this standard:

a) Reduction of fatigue life due to corrosion of structures in corrosive environments beyond normal atmospheric conditions and suitable corrosion protection.

b) Thermal fatigue of structures subject to temperatures beyond 150 °C.

10.1.3 Situations in which no fatigue assessment is required

a) A fatigue assessment may be omitted for ordinary building structures except in the following cases:

- Members supporting lifting appliances or rolling loads.
- Members subjected to repeated stress cycles by vibrating machinery.
- Members subject to wind or water induced oscillations.
- Crowd-induced oscillations

b) A fatigue assessment is not required if the repeatedly applied stresses are insignificant as regards fatigue.

10.2 Fatigue assessment procedures

The aim of designing a structure against the limit state of fatigue failure is to ensure, within an acceptable level of probability, that its performance is satisfactory during its entire design life, such that the structure is unlikely to require repair or to fail by fatigue.

The required safety level shall be obtained by imposing an appropriate fatigue load factor γ_f and a fatigue strength resistance factor γ_r in accordance with 10.7.

The safety verification shall be carried out either in terms of the equivalent stress range by comparing it with the fatigue strength for a given number of stress cycles, or in terms of damage by comparing the applied damage to the limiting damage.

When the constructional detail is defined in the detail classification tables (see 10.5 and A.10.5), the stress range to be used is the nominal stress range, unless otherwise specified in the classification tables.

When the detail differs from a constructional detail defined in the classification tables (see 10.5 and A.10.5) by the presence of a geometric stress concentration effect which should be included, the stress range to be used is the nominal geometric stress range, see 10.2.2.

Careful attention is required in ensuring that the geometric stress is properly evaluated, see 10.4 and that the detail can be considered to be that in the classification tables.

As regards constructional details not included in the detail classifications, 10.5.2 indicates the requirements and fatigue strength curve references to be used in conjunction with the determination of the geometric stress range.

10.2.1 Fatigue assessment based on nominal stress range

10.2.1.1 For variable amplitude loading defined by a design spectrum, the fatigue assessment shall be based on the Palmgren-Miner rule of cumulative damage:

$$D = \sum_i \frac{n_i}{N_i} < 1$$

where

n_i is the number of cycles of stress range $\Delta\sigma_i$ which occurs during the required design life.

N_i is the number of cycles of stress range $\Delta\sigma_i$ to failure, which depends on the detail category.

The cumulative damage assessment shall be based on the relevant slope constants m (see 10.5) for the normal stress range $\Delta\sigma$

Alternatively, the fatigue assessment may be based on an equivalent constant amplitude stress calculation, see A.10.2.1.1.

10.2.1.2 Nominal shear stress ranges, $\Delta\tau$, shall be handled similarly to nominal normal stress ranges, but using a unique slope constant m (see 10.5).

10.2.1.3 Combination of nominal normal and nominal shear stress ranges.

In the case of a combination of normal and shear stresses the fatigue assessment shall consider their combined effects. When applicable, one of the following methods may be used:

a) The maximum principal stress may be used when normal and shear stresses induced by the same loading event are in phase, provided that the planes of the maximum principal stress do not change significantly in the course of a loading event.

b) If, at the same location, normal and shear stresses vary independently, the components of damage using the Palmgren-Miner rule for both normal and shear stresses should be combined according to the following expression:

$$D_{\Delta\sigma} + D_{\Delta\tau} \leq 1$$

where

$D_{\Delta\sigma}$ is the fatigue damage due to normal stresses and calculated according to 10.2.1.1

$D_{\Delta\tau}$ is the fatigue damage due to shear stresses calculated according to 10.2.1.2.

The nominal shear stress may be neglected when its equivalent constant amplitude stress range is less than 15 % of the equivalent nominal normal stress range.

10.2.2 Fatigue assessment based on a geometric stress range

The geometric stress (or "hot spot stress") is defined as the extrapolation of the maximum principal stresses to the weld toe. The maximum values of principal stresses at the weld toe shall be found, investigating various locations around the welded joint or the stress concentration area. The geometric stress takes into account only the overall geometry of the joint, excluding local stress concentration effects due to the weld geometry and discontinuities at the weld toe.

10.3 Fatigue loading

The fatigue loading is to be taken from appropriate ISO or national standards. The loading used for the fatigue assessment should represent a conservative estimate of the accumulated service loading throughout the required design life of the structure. A confidence level of at least 95 % shall be sought for both amplitude and frequency.

Dynamic effects, shall be considered when the dynamic response of the structure contributes to the modification of the design spectrum.

10.4 Fatigue stress spectra

10.4.1 Stress calculation

Stresses shall be determined by an analysis of the structure under fatigue loading, according to elastic theory. Dynamic response of the structure or impact effect shall be considered when appropriate.

a) Stress range for failure in parent material:

Depending upon the fatigue assessment carried out, either nominal stress ranges or geometric stress ranges shall be evaluated.

When using the nominal stress range assessment procedure, a detail shall have a particular category designated if it complies in every respect with the tabulated description. The effect of stress concentrations which are not characteristic of the detail category itself, such as holes, cut-outs, re-entrant corners, etc. shall be taken into account by appropriate stress concentration factors.

Whatever the fatigue assessment procedure used, the effect of stresses arising from joint eccentricity, imposed deformations, secondary stresses due to partial joint stiffness, non-linear stress effects in the post-buckling range (e.g. "breathing" effects in slender webs), shear lag, and prying effect shall be calculated and taken into account when determining the stress at the detail.

b) Stress range for failure in welds:

In load-carrying partial penetration or fillet welded joints, where fatigue failure through the weld throat is being checked, the forces transmitted should be resolved into two stress components, one normal stress-component transverse to the longitudinal axis of the weld and one shear-stress-component parallel to the longitudinal axis of the weld. Their combined damage may be evaluated according to 10.2.1.3, subclause b. The stress-components may be obtained by using the relevant vector components of forces.

10.4.2 Design stress range spectrum

The stress variation or stress history due to a loading event shall be reduced to a stress range spectrum by employing a method of cycle counting.

For a particular detail, the total of all stress range spectra, caused by all loading events, shall be compiled. This compilation results in the design stress range spectrum to be used for the fatigue assessment.

Different components of a structure may have different stress range spectra.

10.5 Fatigue strength

Fatigue strength shall be calculated in accordance with national standards, or as recommended in Appendix A.10.

The expression of the fatigue strength is conveniently presented in the form of log $\Delta\sigma$ - log N-curves, each applicable to typical detail categories. The detail category is designated by a number which represents in MPa a fatigue strength reference value $\Delta\sigma_C$, at 2 million stress cycles.

The fatigue strength curves (normal stress) are expressed by the following equation:

$$\log N = \log a - m \cdot \log \Delta\sigma_R$$

where

$\Delta\sigma_R$ is the fatigue strength
N is the number of stress range cycles
m is the slope constant of the fatigue strength curves
log a is a constant which depends on the slope of the related part of the S-N-curve.

The expression of the fatigue strength curves for shear stress is mathematically equivalent to that of normal stress.

When test data are used to assess a classification category for a particular constructional detail, the 95 % confidence interval of log N at 2 million cycles should be calculated taking into account the standard error of estimate and sample size. The number of data points (not lower than 10) shall be considered in the statistical analysis.

The curves shall be based on representative experimental investigations and as such, include the effects of:

- local stress concentrations due to the weld geometry
- size and shape of acceptable discontinuities
- the stress direction
- residual stresses
- metallurgical conditions
- in some cases, welding process and post weld improvement.

Proper account shall be taken of the fact that residual stresses are low in small scale samples. The resulting fatigue strength curve shall be corrected for the effect of residual stresses occuring in full scale structures.

10.5.1 Definition of fatigue strength curves for classified structural details

The classification of structural details shall be in accordance with national standards, or as recommended in Appendix A.10.

The classification of each part of a structural detail shall account for:

- the directions of the fluctuating stress relative to the detail
- the locations of possible crack initiation
- the geometrical arrangement and proportions of the detail
- the method of manufacture and inspection

In welded details there are several locations at which potential fatigue cracks may initiate, and the detail classification shall take this into account.

10.5.2 Definition of reference fatigue strength curves for non-classified details

For details with geometry not classified in 10.5.1, the fatigue strength shall be-- determined on the basis of reference fatigue strength curves and the geometric stress range.

The reference fatigue strength curves shall be based on fatigue strength tests for relevant structural details, and take account for weld type, weld profile and acceptance criteria for weld defects.

The geometric stress range shall represent the effect of stress concentrations due to the geometry of the detail, such as variation of stiffness within the detail, and load eccentricities.

10.6 Fatigue strength modifications

The influence of mean stress level in non-welded or stress relieved welded details may be accounted for by modifying the compression component of the stress range.

For thicknesses of the parent material that exceed those included in the experimental basis for the fatigue strength curves, a reduced fatigue strength may be obtained based on a model that takes account of the crack propagation in a region with stress gradients.

10.7 Partial safety factors

10.7.1 Partial safety factors for fatigue loading

To take into account uncertainties in the fatigue response analysis, the design stress ranges shall, for the fatigue assessment procedure, include a partial safety factor γ_f.

The factor γ_f covers the uncertainties in estimating:

- the applied load levels
- the conversion of these loads into stresses and stress ranges
- the equivalent constant amplitude stress range from the design stress range spectrum
- the design life of the structure, and the evolution of the fatigue loading within the required design life of the structure.

10.7.2 Partial safety factors for fatigue strength

In order to take into account uncertainties in the fatigue resistance, the characteristic fatigue strength shall be divided by a partial safety factor γ_r.

The factor γ_r covers the uncertainties of the effects of:

- the size of the detail
- the dimension, shape and proximity of the discontinuities
- local stress concentrations due to welding
- variable welding processes and metallurgical effects.

10.7.3 Values of the partial safety factors

Values of the partial safety factors γ_f and γ_r shall be given in the relevant national standards.

Annex A
(informative)

A.6 BASIC VARIABLES

A.6.3 Materials

A.6.3.2 Structural steel

All steel shall be identified in accordance with the requirements of the appropriate ISO or national standard when leaving the mill.

It may be requested to prove the quality and the origin of the steel, if needed.

Any steel which is not in accordance with appropriate ISO or national standard is to be subject to acceptance testing requirements, see ISO 82 and 630.

A.7 ANALYSIS OF STRUCTURES

A.7.1 General

The design strength established on the basis of testing should be greater than the design load by a margin adequate to account for possible deviations of the actual structural element from those tested.

A.7.2 Structural behaviour

The analyses of any structures or structural parts, may be carried out on the basis of one or more of the following

a) Simple construction
b) Continuous construction
c) Semi-continuous construction
d) Experimental verification.

Continuous structures are structures where the beams, girders and trusses are rigidly framed, or are continuous over supports. Connections are generally designed to resist the internal forces which may be computed by assuming that the original angles between intersecting members remain unchanged as the structure is loaded.

In all cases, the details of the members and connections should be such as to realize the assumptions made in the design, without adversely affecting any other part of the structure.

A.7.3 Methods of analysis

A.7.3.2 Elastic analysis

Forces and moments in the structure may be determined by an analysis which assume all members to behave elastically. The yield criterion is the Huber-Hencky-von Mises hypothesis

$$\frac{f_y}{\gamma_r} \geq \sqrt{\sigma^2 x + \sigma^2 y - \sigma_x \sigma_y + 3\tau^2 xy}$$

However, in general it may be accepted that the first yield is exceeded in small local areas, provided that no single stress component exceeds f_y/γ_r.

A.7.3.4 Plastic analysis

When calculating the full strength of a member it is assumed that the material is in a state of yielding in the entire cross-section, whether this is in tension, compression or shear, or in combinations of stresses. Local buckling effects may be handled according to 8.3 or 8.6.

Combinations of moments, shear forces and axial forces may generally be checked by use of relevant interaction formulae. Examples of such interaction formulae are given in A.8.2.1.

Members in structures or portions of structures in which the distributions of moments and forces have been determined by a plastic analysis shall be braced to resist lateral and torsional displacements at, or immediately adjacent to, all hinge locations. The minimum laterally unsupported distance, L_o, from such braced hinge locations to the nearest adjacent and similarly braced point, is given by

$$L_o = 1.3\, i_z \sqrt{\frac{E}{f_y}} \text{ for } 1.0 \geq \frac{M_2}{M_1} \geq 0.5$$

$$L_o = \left(2.0 - 1.4\, \frac{M_2}{M_1}\right) i_z \sqrt{\frac{E}{f_y}} \text{ for } -1.0 \leq \frac{M_2}{M_1} < 0.5$$

where M_2/M_1 is equal to the ratio of the smaller to the larger moment at opposite ends of the unbraced length in the plane of bending, positive when the member is bent in single curvature and negative when the member is bent in double curvature.

i_z is the lateral radius of gyration for the entire section.

The necessary control of the bracing may be done according to A.8.5.4. The bracing must be designed to be effective when the structure reaches its ultimate limit.

Where the sequence of formation of hinges can be predicted with certainty, bracing is not required at the location of the last hinge to form in the relevant failure mechanism.

A.8 ULTIMATE LIMIT STATES

A.8.2 Resistance of structural members

A.8.2.1 Calculation of sections which may reach full plastification, i.e. class 1 and 2 sections, can be done by any rational stress distribution which is in consistency with the laws of equilibrium. Alternatively, interaction type of formulae may be used, calculating the different forces and or moments and relate them to their fully plastified capacities. Simplified formulae are given in A.8.2.1.1 - A.8.2.1.5.

A.8.2.1.1 Axial tension:

The axial tensile resistance N_d of a member is by the lesser of

$$N_{yd} = A_g \cdot f_y \frac{1}{\gamma_{ry}}$$

or

$$N_{ud} = A_n \cdot f_u \frac{1}{\gamma_{ru}}$$

Where ductile behaviour is desired, $N_{ud} > N_{yd}$, and therefore

$$\frac{A_n}{A_g} \geq \frac{\gamma_{ru}}{\gamma_{ry}} \frac{f_y}{f_u}$$

where $\gamma_{ru} > \gamma_{ry}$

A.8.2.1.2 Compression resistance:

The axial compressive resistance, N_{rd}, developed by a member, not subject to local instability, is given by

$$N_{rd} = \frac{N_r}{\gamma_r} = \frac{A f_y}{\gamma_r}$$

where A is the area available to resist the compressive forces.

If the member is subject to local instability for the whole or parts of the section, i.e. the member has a class 4 section, the compressive resistance of the section will be reduced, see A.8.3.2 and A.8.4.4.3.

A.8.2.1.3 Shear resistance:

For members with a web where no premature buckling will occur, the shear resistance of the web is given by:

$$V_{cd} = \frac{\tau_y}{\gamma_r} A_w$$

where A_w is the relevant web area and τ_y is the shear yield stress.

The condition that premature shear buckling will not occur is met if the depth over thickness does not exceed $2.2 \sqrt{E/f_y}$.

For plastic design τ_y may be taken as

$$\tau_y = 0.6\ f_y$$

and for elastic design including strain hardening τ_y may be taken as

$$\tau_y = 0.66\ f_y$$

The upper limit of

$$\tau_u \leq 0.58\ f_u$$

should generally be adopted, where the resistance is determined on the basis of the net section.

The calculations of shear resistance must include the effects of possible web cut-outs.

Members for which premature shear buckling may occur should be designed according to A.8.6.3.

A.8.2.1.4 Uniaxial moment resistance:

The uniaxial moment resistance of a member which is adequately supported to prevent any lateral torsional buckling effects, is for sections of class 1 and 2 given by

$$M_{rd} = \frac{f_y}{\gamma_r} W_p = \frac{M_p}{\gamma_r}$$

and for sections of class 3 given by

$$M_{rd} = \frac{f_y}{\gamma_r} W = \frac{M_y}{\gamma_r}$$

For sections of class 4 the resistance calculations must adequately include the effects of local buckling. This may either be done by adopting the concept of a reduced effective cross-section whose section modulus is W_e, see A.8.6, in which case the moment resistance is given by

$$M_{rd} = \frac{f_y}{\gamma_r} W_e$$

or by adopting the concept of a reduced effective yield strength κf_y, in which case the moment resistance is given by

$$M_{rd} = \frac{\kappa f_y}{\gamma_r} W$$

Both concepts must be sufficiently correlated to experiments and/or numerical simulations. For the former of the two concepts, A.8.6 may be adopted.

A.8.2.1.5 Combination of forces:

If a member is subjected to a combination of forces and it is adequately supported to prevent any lateral torsional buckling effects, then its resistance may be checked by an appropriate interaction formula, which may either be based on the theory of plasticity and finally verified by experiments, or it may be based on experiments alone.

For members with an I-section of class 1 or 2, the resistance may be checked according to the following sets of formulae:

a) The combination of strong axis moment and shear may be checked by reducing the plastic moment resistance according to

$$M_{red} = M_f + M_w \left(1 - \left(\frac{V}{V_d}\right)^2\right)$$

where M_f and M_w are the plastic moment resistance of the flanges and the web, respectively, or alternatively according to

$$M_{red} = M_{rd} \left(1.4 - 0.6 \frac{V}{V_d}\right) \leq M_{rd})$$

b) The combination of axial force and biaxial moment, may be checked according to

$$\frac{N}{N_{rd}} + 0.85 \frac{M_y}{M_{dy}} + 0.60 \frac{M_z}{M_{dz}} \leq 1$$

$$\frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1$$

where y and z are the strong and weak axes of the cross-section, respectively.

For members with a cross-section of class 3, stresses should be calculated according to simple theory of elasticity and the resistance may be checked according to the yield criterion given in A.7.3.2. If the member is subjected to a combination of axial force and biaxial moment, the linear interaction formula

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1$$

may be adopted.

For members with a cross-section of class 4, the recommentations given in A.8.6.4 may be adopted.

A.8.3 Classification of cross sections

A.8.3.1 General

In general the width-thickness ratio of a compression element must be defined, as the boundary conditions of a plate are dependant on whether the edges are free, hinged or considered to be fixed, see Fig. A.8.3.1. A single fillet weld will act as a hinge, whereas a double fillet weld may give restraining effects.

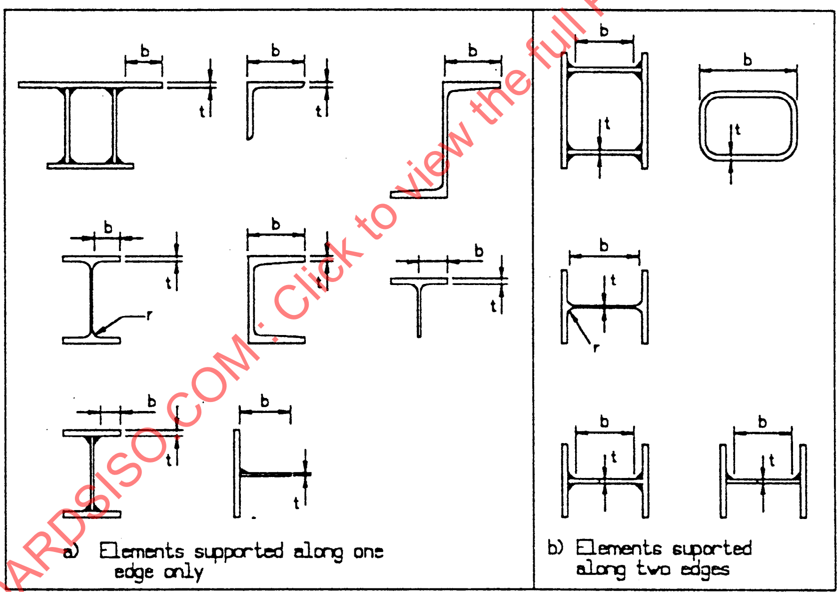


Fig. A.8.3.1

For elements supported along one edge only, parallel to the direction of the compressive force, the width shall be taken as follows:

a) For plates and flanges of I-shapes, the width (b) is the distance from the free edge to the first row of fasteners or the toe of fillet welds or the transition curve to the web.

b) For legs of angles, flanges of channels and zees, and stems of tees, the width (b) is the full nominal dimension.

For elements supported along two edges parallel to the direction of the compressive force, the width shall be taken as follows:

a) For flange or diaphragm plates in built-up sections the width (b) is the distance between adjacent lines of fasteners or welds.

For elements supported by two welds on each side the width is the distance between the weld toes. For rolled sections the width is the distance between the transition to the flanges.

b) For flanges of rectangular hollow structural sections the width (b) is the width measured on the outside of the section.

The thickness of elements is the nominal thickness. For tapered flanges of rolled sections, the thickness is the nominal thickness halfway between a free edge and the corresponding face of the web.

A.8.3.2 Definitions of classes

Class 2 I-sections in continuous beams may be calculated as if they were class 1 sections provided the material yield strength f_y is replaced by a reduced effective yield strength, f_{ye}. For a free flange f_{ye} may be calculated as

$$f_{ye} = 0{,}102\,E\,\left(\frac{t}{b}\right)^2 \leq f_y$$

and similarly, for a web in pure bending and with hinged boundaries the effective yield strength may be calculated as

$$f_{ye} = 6{,}25\,E\,\left(\frac{t}{b}\right)^2 \leq f_y$$

Based on the effective yield strength f_{ye}, effective plastic resistances may be calculated.

For class 3 sections the general requirement is that the section shall be able to attain the yield stress in its most compressed fibre prior to local buckling or distortion of the cross section. According to the methods given in 8.6 and A.8.6 this implies that any plate or part of the cross-section must comply with the width-thickness requirement

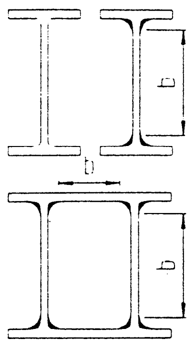
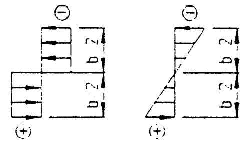
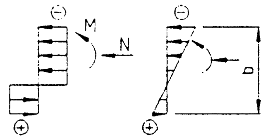
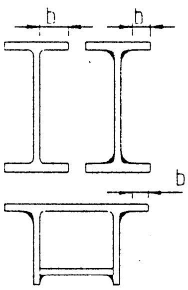
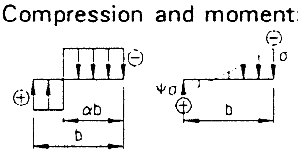
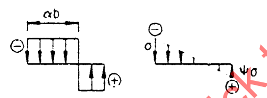
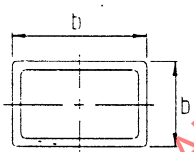
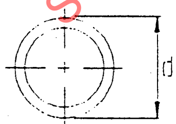
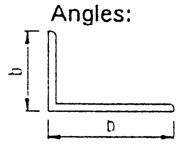
$$\frac{b}{t} \leq \frac{0.71}{1.05}\sqrt{k}\sqrt{\frac{E}{f_y}} = 0.676\sqrt{k}\sqrt{\frac{E}{f_y}}$$

The factor k depends on the boundary conditions, see 8.6. The edge condition comprising one fillet weld is equivalent to a hinge. In Fig. A.8.3.1 some examples are given of how to define the b/t relation.

A.8.3.3 Maximum width-thickness ratios of elements subjected to compression and/or bending

Recommended values for the maximum width-thickness ratios of elements subjected to compression and bending are given in Table A.8.3.3.

Table A.8.3.3 Classification of cross-sections (*NA = Not Applicable*)

Cross section element	Stress distribution in the element	Medium b/t - ratio Class 1	Class 2	Class 3	Class 4
Webs of I-sections. Webs or flanges of welded box sections 	Axial compression	NA	NA	$1{,}4\sqrt{E/f_y}$	Note 1
	Moment:  Class 1 & 2 Class 3	$2.5\sqrt{\frac{E}{f_y}}$	$3.8\sqrt{\frac{E}{f_y}}$	$4.2\sqrt{\frac{E}{f_y}}$	
	Compression and moment:  Class 1 & 2 Class 3	$2.5\,K_1\sqrt{\frac{E}{f_y}}$	$3.8\,K_2\sqrt{\frac{E}{f_y}}$	$4.2\,K_3\sqrt{\frac{E}{f_y}}$	
Flanges of I-sections. Free flanges of welded box sections 	Compression or strong axis moment	$0.32\sqrt{E/f_y}$	$0.37\sqrt{E/f_y}$	$0.45\sqrt{E/f_y}$	
	Compression and moment: 	$\frac{0.32}{\alpha\sqrt{\alpha}}\sqrt{\frac{E}{f_y}}$	$\frac{0.37}{\alpha\sqrt{\alpha}}\sqrt{\frac{E}{f_y}}$	$0.69\,K_4\sqrt{\frac{E}{f_y}}$	
	 Class 1 & 2 Class 3	$\frac{0.32}{\alpha\sqrt{\alpha}}\sqrt{\frac{E}{f_y}}$	$\frac{0.37}{\alpha\sqrt{\alpha}}\sqrt{\frac{E}{f_y}}$	$0.69\,K_5\sqrt{\frac{E}{f_y}}$	
Rectangular hollow sections: 	Compression (flange)	$1.1\sqrt{\frac{E}{f_y}}$	$1.4\sqrt{\frac{E}{f_y}}$	$1.7\sqrt{\frac{E}{f_y}}$	
	Moment (web)	$2.5\sqrt{\frac{E}{f_y}}$	$3.8\sqrt{\frac{E}{f_y}}$	$4.3\sqrt{\frac{E}{f_y}}$	
Circular hollow sections: 	Compression and/or moment	$0.065\frac{E}{f_y}$	$0.090\frac{E}{f_y}$	$0.115\frac{E}{f_y}$	
Angles: 	Compression	NA	NA	$0.45\sqrt{\frac{E}{f_y}}$	

$K_1 = 1 - 0.39\ N/N_p$ $K_2 = 1 - 0.63\ N/N_p$ $K_3 = 1 - 0.67\ N/N_p$	N/N_p refers to the fully cross section for doubly symmetric sections
$K_4 = [0.425 - 9.1(\psi+1) + 10.4(\psi+1)^2]^{1/2}$ $K_5 = [0.57 + 0.2\psi + 0.07\psi^2]^{1/2}$	ψ is positive as indicated above

Note 1: Class 4 are governed by A.8.8 or alternatively A.8.4.4.3

A.8.4 Flexural buckling

A.8.4.1 Effective length

The effective buckling length of a member, may vary considerably due to the end restraint conditions. It should generally be acknowledged that a 100% effective end restraint is difficult to obtain. The effective buckling length may be determined by simplified methods, or based on theory of elasticity.

A.8.4.2 Slenderness

Relative slenderness is introduced to give non-dimensional formulae and diagrams, i.e. diagrams which can be used independent of the yield strength of the steel.

$$\bar{\lambda} = \frac{\lambda}{\lambda_c} \quad \text{where} \quad \lambda_c = \pi\sqrt{\frac{E}{f_y}}$$

A.8.4.3 Compression resistance

For cross-sections of class 1, 2 or 3, see Table A.8.3.3, the resistance will be

$$N_{cd} = \frac{1}{\gamma_r} f_c \cdot A_g = f_{cd} \cdot A_g$$

where f_c is the buckling strength.

For cross-sections of class 4 the resistance in compression may because of local buckling be handled as given in A.8.4.4.3.

A.8.4.4 Determination of f_c

A.8.4.4.1 Alternative 1:

The buckling strength f_c may be obtained from Fig. A.8.4.4.a, where f_c is presented as a function of the relative slenderness $\bar{\lambda}$. The buckling strength curves of Fig. A.8.4.4.a are the result of an extensive test program involving tests of columns of various cross-sections.

For members with cross-sections as shown in Fig. A.8.4.4.b, f_c is determined from the appropriate curve. For cross-sections not shown in Fig. A.8.4.4.b, curve c may be used.

The values of f_c presented in Fig. A.8.4.4.a may with a very good accuracy be found from the expression

$$\frac{f_c}{f_y} = \left[B + (B^2 - \bar{\lambda}^2)^{0.5}\right]^{-1}$$

where

$$B = 0.5\left[1 + \alpha\,(\bar{\lambda} - \bar{\lambda}_0) + \bar{\lambda}^2\right]$$

α represents the effects of initial out-of-straightness, load eccentricity and residual stresses.

$\bar{\lambda}_0$ represents the relative slenderness, below which no instability will occur due to strain hardening effects.

Unless the curves of Fig. A.8.4.4.a are adopted, values of α and $\bar{\lambda}_0$ should be given in national standards.

For the curves in Fig. A.8.4.4.a the following values may be adopted:

- buckling curve a: α = 0.21, $\bar{\lambda}_0$ = 0.2
- buckling curve b: α = 0.34, $\bar{\lambda}_0$ = 0.2
- buckling curve c: α = 0,49, $\bar{\lambda}_0$ = 0.2

For members with practically no residual stresses α = 0.13 and $\bar{\lambda}_0$ = 0.2 may be adopted.

For members with residual stresses of prevailing importance to its behaviour, e.g. thick-walled heavy columns, α = 0.76 and $\bar{\lambda}_0$ = 0.2 may be adopted.

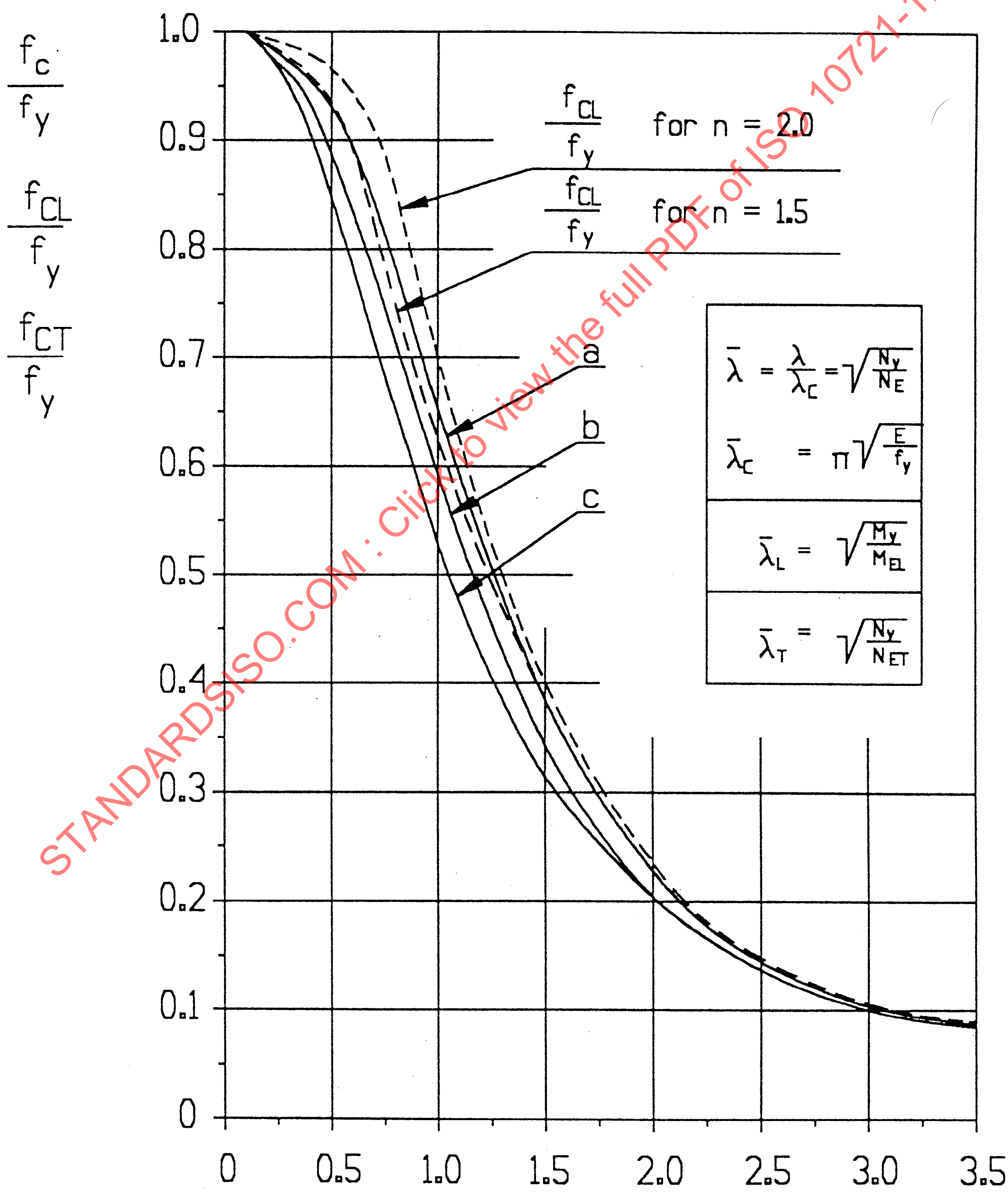


Fig. A.8.4.4.a Buckling curves, alternative 1

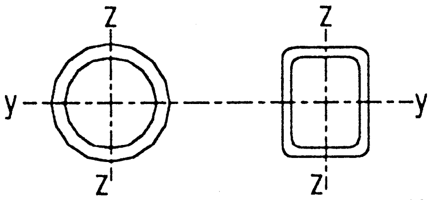
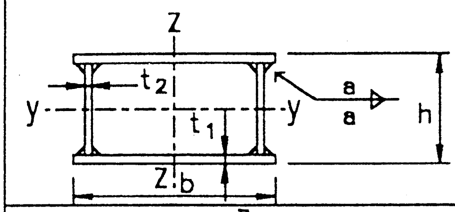
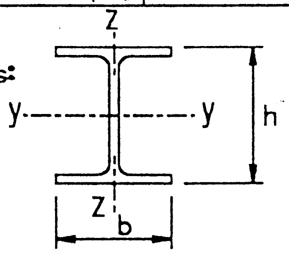
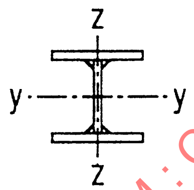
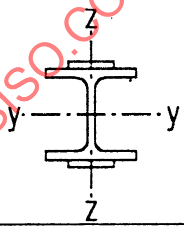
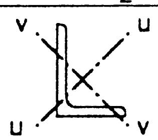
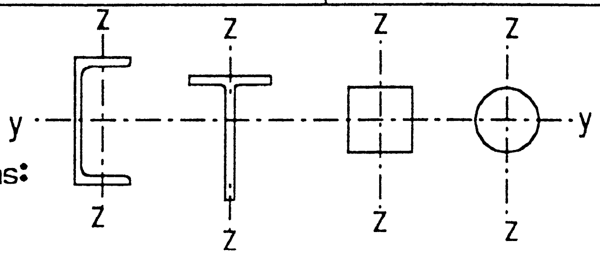
Cross section:	Condition requirement	Buckling perpendicular to the axis	Buckling curve
Hollow sections: 	Hot formed, or cold form and stress relieved	y-y or z-z	a
	Cold formed (f_y based on stub column testing)	y-y or z-z	c
Welded box sections: 	Stress relieved	y-y or z-z	a
	As welded (except as below)	y-y or z-z	b
	Thick welds ($a > \frac{t}{2}$), $\frac{b}{t} < 30$	y-y	c
	Thick welds ($a > \frac{t}{2}$), $\frac{h}{t} < 30$	z-z	c
Rolled I-sections: 	$\frac{h}{b} \leq 1.2$	y-y	a
		z-z	b
	$\frac{h}{b} \leq 1.2$	y-y	b
		z-z	c
Welded I-sections: 	Stress relieved	y-y	a
		z-z	b
	Flame cut flanges	y-y or z-z	b
	Rolled flanges	y-y	b
		z-z	c
Re-enforced I-sections: 	Rolled I-sections with welded flange plates	y-y	b
		z-z	a
L-sections: 	Generally	u - u or v - v	c
	Hot-dip galvanized		b
U-, T- and solid sections: 		y-y or z-z	c

Fig. A.8.4.4.b Relation between cross section and appropriate buckling curves, alternative 1.

A.8.4.4.2 Alternative 2:

The buckling strength f_c may be obtained from a single mean-value-curve, e.g. based on the expression

$$\frac{f_c}{f_y} = e^{-\kappa\bar{\lambda}2} \quad \text{for} \quad \bar{\lambda} \le \bar{\lambda}_1$$

$$\frac{f_c}{f_y} = \frac{\eta}{\bar{\lambda}^2} \quad \text{for} \quad \bar{\lambda} > \bar{\lambda}_1$$

The values of κ, η and $\bar{\lambda}_1$ should be given in national standards.

This alternative mean-value curve may necessitate a redefiniation of the resistance factors, see 6.5, as compared to Alternative 1, given in A.8.4.4.1. All relevant effects may, however, be adequadely taken into account by an appropriate choice of the resistance factor and the values:

$\kappa = 0.419$

$\eta = 0.877$

$\bar{\lambda}_1 = 1.5$

A.8.4.4.3 The determination of f_c for members with a cross section of class 4 must include the effects of local buckling.

This can be obtained by replacing the actual yield strength f_y with a reduced yield strength which is equal to the lowest local plate buckling strength f_{cp} of the cross-section. The buckling strength f_c for such a member may then be calculated according to either of the two alternatives given in A.8.4.4.1 and A.8.4.4.2, only with f_y replaced by f_{cp}, and the buckling resistance of the member may be obtained as if its cross-section is of class 3.

Alternatively, the determination of f_c for a member with a cross-section of class 4 may be handled by adopting the concept of a reduced effective cross-section, which includes all relevant effects, and is obtained from experiments and/or numerical simulations. It can generally be expressed as a function of the effective cross-section and the buckling parameters of the member. The buckling resistance of the member may finally be calculated as if its cross section is of class 3.

A.8.4.4.4 For cold formed sections, neither normalized nor stress relieved, the residual stresses and the variation of material properties due to cold forming will affect the buckling resistance of such members. As given in Fig. A.8.4.4.b, buckling curve c should be adopted for members with cold formed hollow sections, provided the determination of f_y is based on stub column testing.

A.8.4.4.5 For L-sections buckling curve b should be adopted, unless the members is hot-dip galvanized, in which case the buckling curve a may be adopted.

For a member with an L-section, connected in one leg only, the support conditions will affect its buckling resistance. This may be taken into account by adopting the concept of replacing the actual relative slenderness $\bar{\lambda}$ (based on the minimum radius of gyration) with an effective relative slenderness $\bar{\lambda}_e$, which may be calculated according to the following:

a) The angle is connected with one bolt at each end:

$$\bar{\lambda}_e = 0.60 + 0.57\bar{\lambda} \quad \text{if} \quad \bar{\lambda} \leq 1.41$$

$$\bar{\lambda}_e = \bar{\lambda} \quad \text{if} \quad \bar{\lambda} > 1.41$$

b) The angle is connected with two bolts or welded, and the adjacent chords are not loaded to their full capacity:

$$\bar{\lambda}_e = 0.60 + 0.57\bar{\lambda} \quad \text{if} \quad \bar{\lambda} \leq 1.41$$

$$\bar{\lambda}_e = 0.35 + 0.75\bar{\lambda} \quad \text{if} \quad 1.41 \leq \bar{\lambda} \leq 3.5$$

A.8.4.5 Compression members subjected to moments

Examples of interaction equations considered to meet the requirements of 8.4.5 are given in A.8.4.5.1 and A.8.4.5.2.

A.8.4.5.1 Alternative 1:

$$\frac{N}{N_{cd}} + \frac{k_yM_y + Ne_y}{M_{dy}} + \frac{k_zM_z + Ne_z}{M_{dz}} \leq 1$$

where:

y and z indicate the strong and the weak axis of the cross-section, respectively.

N_{cd} = the lesser of N_{cy} and N_{cz} if lateral torsional buckling is prevented.
N_{cz} if lateral torsional buckling is not prevented.

N_{cy}, N_{cz} = For cross-sections of class 1, 2 or 3, see Clause A.8.4.3.
For cross-sections of class 4, the concept of a reduced effective cross-section, according to clause A.8.4.4.3, should be adopted.

M_{dy} = According to Clause 8.2.1 and A.8.2.1.4 if lateral torsional buckling is prevented.
According to Clause 8.5.2 and A.8.5.3 if lateral torsional buckling is not prevented.

M_{dz} = According to clause 8.2.1 and A.8.2.1.4

k_y = $1 - \mu_y N/(\gamma_r N_{cy}) \leq 1.5$ if lateral torsional buckling is prevented
$1 - \mu_L N/(\gamma_r N_{cz}) \leq 1.0$ if lateral torsional buckling is not prevented

k_z = $1 - \mu_z N/(\gamma N_{cz}) \leq 1,5$

μ_y = $\bar{\lambda}_y (2\omega_y - 4) + (W_{py} - W_y)/W_y \leq 0.9$ for Class 1 and 2 sections
$\bar{\lambda}_y (2\omega_y - 4) \leq 0.9$ for Class 3 and 4 sections

μ_z = $\bar{\lambda}_z (2\omega_z - 4) + (W_{pz} - W_z)/W_z \leq 0.9$ for Class 1 and 2 sections
$\bar{\lambda}_z (2\omega_z - 4) \leq 0.9$ for Class 3 and 4 sections

μ_L = $0.15\,(\bar{\lambda}_z \omega_y - 1) \leq 0.9$

ω_y, ω_z = See Fig. A.8.4.5.1.

e_y, e_z = Not applicable (i.e equal to zero) for cross-sections of class 1,2 and 3.

For cross-sections of Class 4 e_y and e_z are the shift of relevant neutral axis according to an effective cross-sectional calculation due to uniform compression, see Clause A.8.4.4.3.

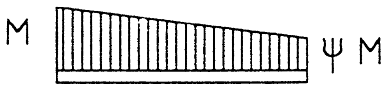
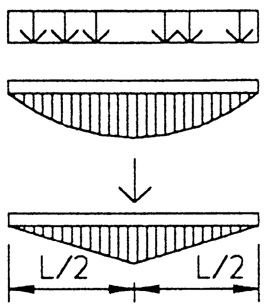
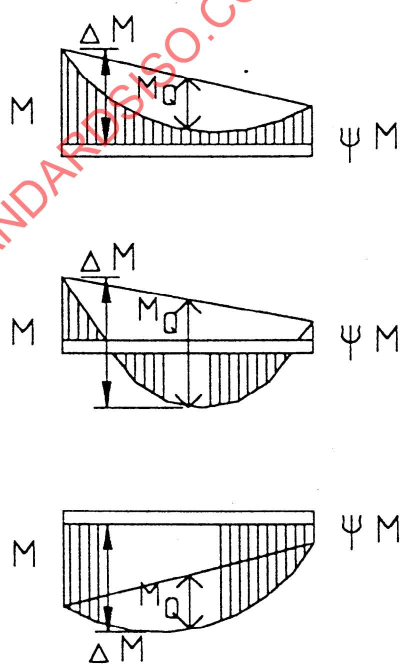
Moment diagram	ω
End moments alone:  $-1 \leq \psi \leq 1$	$\omega_M = 1.8 - .0.7\,\psi$
Transverse loads alone: 	$\omega_Q = 1.3$ $\omega_Q = 1.4$
Combined end moments and transverse loads: 	$\omega_{MQ} = \omega_M + \dfrac{M_Q}{\Delta M}(\omega_Q - \omega_M)$ where M_Q = /max M/ due to transverse load alone ΔM = { /max M/ for moment diagrams without change of sign; /max M/+/min M/ for moment diagrams with change of sign }

Fig. A.8.4.5.1 Moment coefficient, ω.

A.8.4.5.2 Alternative 2:

Members other than Class 1 sections of I-shaped members, subjected to bending moments and an axial compressive force may be checked by the interaction formulae

a) Cross-sectional resistance

$$\frac{N}{N_{cy}} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

b) Overall member resistance

$$\frac{N}{N_{cz}} + \frac{\beta_y M_y}{M_{dy}(1 - \frac{N}{N_{Ey}})} + \frac{\beta_z M_z}{M_{dz}(1 - \frac{N}{N_{Ez}})} \leq 1.0$$

except when $M_z = 0$, take $N_c = N_{cy}$

c) Lateral torsional buckling

$$\frac{N}{N_{cz}} + \frac{\beta_y M_y}{M_{Ld}(1 - \frac{N}{N_{Ey}})} + \frac{\beta_z M_z}{M_{dz}(1 - \frac{N}{N_{Ez}})} \leq 1.0$$

with

$$\frac{\beta_y}{1 - \frac{N}{N_{Ey}}} \geq 1.0$$

Class 1 sections of I-shaped members subjected to bending moments and an axial compressive force may be checked by the interaction formulae

a) Cross-sectional resistance

$$\frac{N}{N_{rd}} + \frac{0.85M_y}{M_{dy}} + \frac{0.60M_z}{M_{dz}} \leq 1.0$$

$$\frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

b) Overall member resistance

$$\frac{N}{N_{cz}} + \frac{0.85\beta_y M_y}{M_{dy}(1 - \frac{N}{N_{Ey}})} + \frac{0.60\beta_z M_z}{M_{dz}(1 - \frac{N}{N_{Ez}})} \leq 1.0$$

except when $M_z = 0$ take $N_c = N_{cy}$

c) Lateral torsional buckling resistance, if applicable

$$\frac{N}{N_z} + \frac{\beta_y M_y}{M_{Ld}(1 - \frac{N}{N_{Ey}})} + \frac{\beta_z M_{yz}}{M_{dz}(1 - \frac{N}{N_{Ez}})} \leq 1.0$$

with

$$\frac{\beta_y}{1 - \frac{N}{N_{Ey}}} \geq 1.0$$

N, M_y, M_z are the design values of the action effects and M_y, M_z include 2nd-order translation moments if applicable.

$N_{rd} = f_y A_g / \gamma_r$

N_{cy}, N_{cz} are based on L/i_y and L/i_z respectively and include γ factors.

N_{Ey}, N_{Ez} are Euler buckling loads based on L/i_y and L/i_z respectively.

M_{dy}, M_{dz} are the relevant cross-sectional moment resistances depending on the section classification.

M_{dL} is the lateral torsional buckling resistance based on the moment diagram that exists (i.e. includes non-uniform moment effects).

Unless a more accurate analysis is carried out the equivalent moment, $\beta_y M_y$ or $\beta_z M_z$, shall be taken as:

a) For members subject to end moments only:

$$\beta M = \beta_1 M = 0.6\ M_1 - 0.4 M_2 \geq 0.4\ M_1$$

where M_1 is the larger end moment and M_2 is the smaller end moment at the opposite end of the unbraced length and is taken as positive for double curvature and negative for single curvature.

b) For members subject to moments due to transverse load only:

$$\beta M = \beta_o M_o = M_o$$

where M_o is the maximum moment due to transverse load in the unbraced length.

c) For members subject to both end moments and transverse loads, where M_1 and M_o are as defined in a and b above

(i) $\beta M = \beta_1 M + \beta_o M_o = \beta_1 M + M_o$

when M_1 and M_o cause curvature in the same direction,

(ii) $\beta M = \beta_1 M = 0.6 M_1 - 0.4 M_2$

when M_1 and M_o cause curvature in opposite directions and $|M_o| \leq |M_1|$,

(iii) $\beta M = |M_o| - \beta_1 |M_1|$

when M_1 and M_o cause curvature in opposite directions and $|M_o| > |M_1|$

A.8.4.6 Buckling of built-up members

Built-up members with principal axis (y) equal to the principal axis (y) of the individual members, see Fig. A.8.4.6.a, shall be checked for buckling about this axis according to A.8.4.1 - A.8.4.5.

A.8.4.6.2 Built-up members for which none of the principal axes (y or z) coincide with the principal axis of the individual members, may be checked for buckling about the principal axis according to A.8.4.1 - A.8.4.5, provided λ is replaced by λ_i.

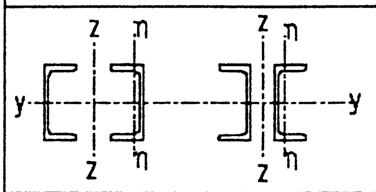
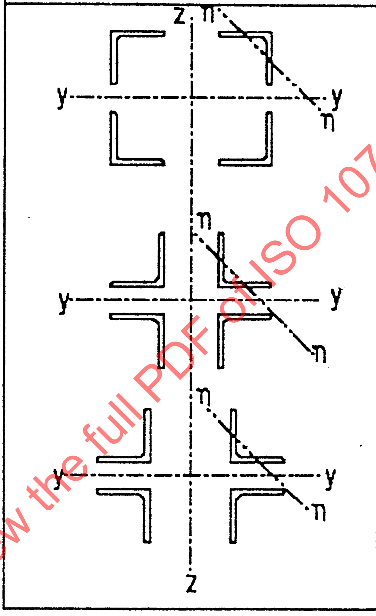
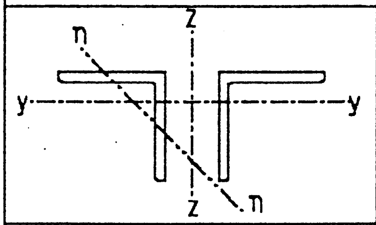
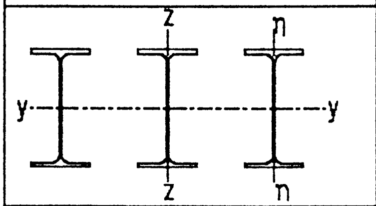
Cross section	n y-y	n z-z	Cross section	n y-y	n z-z
	0	2		2	2
	0	2			
	0	3			

Fig. A.8.4.6.a Coefficient n for various built-up sections

For batten columns λ_i is given by

$$\lambda_i = \sqrt{\lambda^2 + \varphi \cdot \frac{n}{2} \cdot \lambda_s^2}$$

where n is given in Fig. A.8.4.6.a.

λ is the slenderness of the entire member and λ_s is the slenderness of an individual member about its principal axis, i.e. the η-η axis in Fig. A.8.4.6.a.

ϕ = 1 for welded or slip-critical connections,
ϕ = 1.3 for riveted or reamed bolt connections.

For other bolted connections larger values are to be used.

$\lambda_s = s_l/i_l$, where s_l is the center spacing of the reinforcing plates, but not more than the clearance between them plus 100 mm, see Fig. A.8.4.6.b.

For laced columns g_i is given by

$$\lambda_i = \Psi \cdot \lambda$$

For welded and slip critical connections, Ψ = 1.1.
For riveted or reamed bolt lacing, Ψ = 1.2.

For batten and laced columns with $\lambda > 50$, it is required that $\lambda_s < 0.5\,\lambda$.

For built up members with individual member spacing corresponding to the gusset thickness, see Fig. A.8.4.6.a, $\lambda_i = \Psi \cdot \lambda$ when the member, aside from its battens or lacing, has riveted, reamed bolted or welded intermediate ties spaced no more than $15 i_i$ apart.

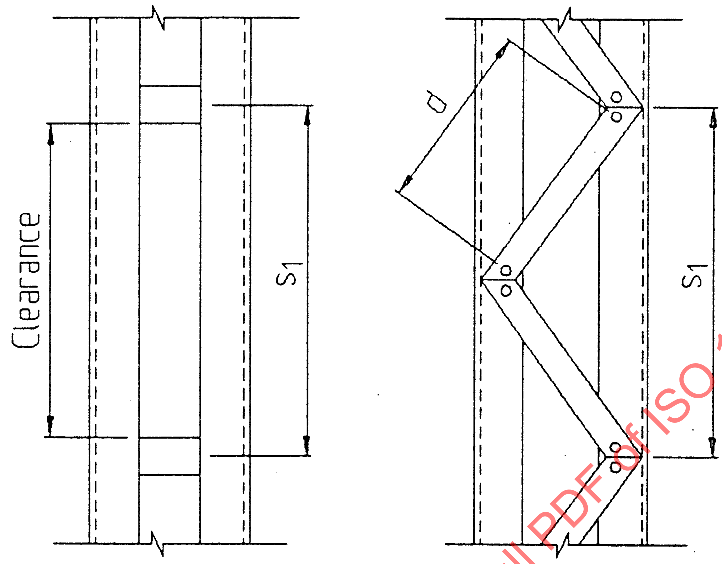


Fig. A.8.4.6.b Spacing of built up members

A.8.4.6.3 The members (main members and battens or lacing) shall be designed for the notional shear force

$$V_i = 1.2\,V \geq 0.012\,\frac{f_y}{\gamma_r}\,A_g$$

and the bending moment due to V_i in the main member and the battens.

A_g is the total cross sectional area of the member and V is the external shear force.

For simplicity V_i may be taken as constant along the member. Alternatively, the effects of its variation may be considered by more accurate calculations.

A.8.5 Torsional and lateral torsional buckling

A.8.5.3 Buckling strength f_{cT} and f_{cL}

The torsional and the lateral torsional buckling resistance of members may be handled similarly to the more simple case of compression members, i.e. the elastic buckling strength resultants and the material strength are employed to express the resistance of the member.

The buckling strength for torsional or flexural-torsional buckling, f_{cT}, may be taken from 8.4.4 and A.8.4.4.1, applying

- curve a for rolled sections, and
- curve c for welded sections.

The buckling strength for lateral torsional buckling of members with cross sections of class 1, 2 or 3 may be taken as

$$f_{cL} = f_y \,(1 + \bar{\lambda}_L^{2n})^{-1/n}$$

where n = 2 for hot rolled and lightly welded sections, n = 1.5 for other welded sections, and where λ_L is defined in A.8.5.3.2.

For welded sections with flame cut flanges n = 1.5 may be conservative.

A.8.5.3.1 The relative slenderness parameter for torsional buckling is given by

$$\bar{\lambda}_T = \sqrt{\frac{N_y}{N_{ET}}}$$

where $N_y = f_y \cdot A_g$ and N_{ET} is the elastic torsional or flexural-torsional buckling force.

The elastic flexural-torsional buckling force N_{ET} is generally given by the solution of the following cubic equation

$$(N_{Ex} - N_{ET})(N_{Ey} - N_{ET})(N_{Ez} - N_{ET})\, i_p^2 - y_s^2\, N_{ET}^2(N_{Ey} - N_{ET}) - x_s^2 N_{ET}^2(N_{Ez} - N_{ET}) = 0$$

where

N_{Ey}, N_{Ez} are the effective elastic buckling forces for buckling about the y- and z-axis, respectively, see 8.4.1.

y_s, z_s are the shear centre coordinates with respect to the centroid of the cross-section

$i_p^2 = (I_y + I_z)/A$ is the polar radius of gyration

$N_{Ex} = (GI_t + \dfrac{\pi^2 EC_w}{L_t^2}) / i_p^2$ is the torsional buckling force

L_t is the effective elastic torsional buckling length.

For doubly symmetric cross-sections $y_s = z_s = 0$, and then $N_{ET} = N_{Ex}$ (provided $N_{Ex} < N_{Ey}$ and N_{Ez}).

For singly symmetric cross-sections, e.g. with $y_s = 0$, the elastic flexural-torsional buckling force is given by

$$N_{ET} = \frac{N_{Ez}}{2(1 - y_s^2/i_p^2)} \left[1 + \frac{N_{Ex}}{N_{Ez}} - \sqrt{(1 - \frac{N_{Ex}}{N_{Ez}})^2 + 4(\frac{y_s}{i_p})^2 \frac{N_{Ex}}{N_{Ez}}} \right]$$

A.8.5.3.2 The relative slenderness parameter for lateral torsional buckling is given by

$$\bar{\lambda}_L = \sqrt{\frac{M_y}{M_{EL}}}$$

where $M_y = f_y\, W$ and M_{EL} is the elastic lateral torsional buckling moment.

For beams with doubly symmetric cross-sections and with actions in the shear center, M_{EL} is given by

$$M_{EL} = \varphi \; \frac{\pi}{L} \sqrt{EI_y GI_T} \sqrt{1 + \frac{\pi^2}{L^2} \; \frac{EC_w}{GI_T}}$$

where ϕ depends on the actions and the support conditions of the member, see Fig. A.8.5.3.2.b.

For beams with monosymmetric cross-sections, i.e. sections with weak axis symmetry, and with actions outside the shear center, M_{EL} is given by

$$M_{EL} = \varphi \; \frac{\pi^2 EI_z}{L^2} \left[\sqrt{L^2 \frac{GI_T}{\pi^2 EI_z} + \frac{C_w}{I_z} + \left(\frac{\Psi_y + a}{2}\right)^2} - \frac{\Psi_y + a}{2} \right]$$

where c_x is a cross sectional parameter given by

$$\psi_y \; \frac{1}{I_x} \int_A z \, (y^2+z^2) \; dA \; - \; 2z_s$$

and a is the distance between the cross-sectional center of gravity and the point of action, see Fig. A.8.5.3.2.a.

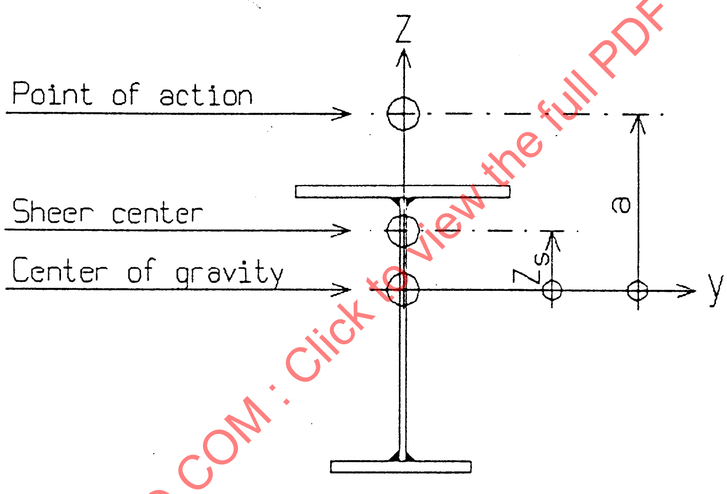


Fig. A.8.5.3.2.a Cross-sectional definitions.

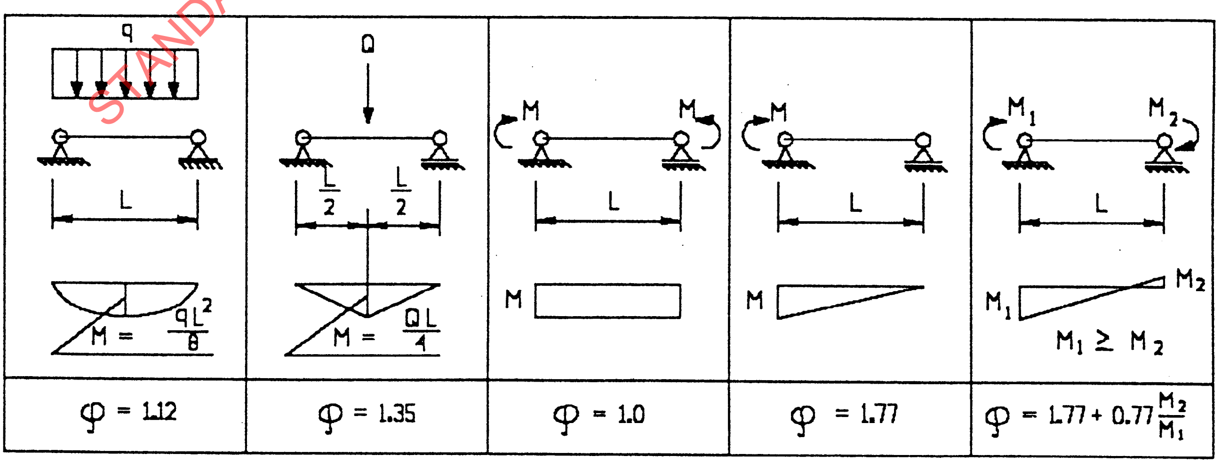


Fig. A.8.5.3.2.b - ϕ-values for some common cases.

A.8.5.3.3 For a member with a cross-section of class 4 the local buckling effects may be handled by replacing the actual yield strength f_y with a reduced yield strength which is equal to the lowest local plate buckling strength f_{cp} of the cross section. The torsional and the lateral torsional buckling resistance of the member may then be calculated according to the recommendations for class 3 sections, only with f_y replaced by f_{cp}. The elastic local buckling strength f_{cp} may be obtained from A.8.6.2.

The torsional and the lateral torsional buckling resistance of a member with a cross-section of class 4 may also be obtained by employing the concept of a reduced effective cross-section, i.e. the gross section resistance is reduced by a factor k such that the torsional buckling resistance may be expressed as

$$N_{Td} = \kappa_t f_y A_g / \gamma_r$$

and the lateral torsional buckling resistance may be expressed as

$$M_{Ld} = \kappa_L f_y W / \gamma_r$$

where the reduction factors κ_T and κ_L are obtained from experiments and/or numerical simulations. They can generally be expressed as functions of the effective cross-section and the buckling parameters of the member.

A.8.5.3.4 The inelastic lateral torsional buckling resistance of columns and beams may be determined from inelastic lateral torsional buckling curves and that of beam-columns from interaction equations based on such curves as are given in national standards. Such curves shall provide sufficiently good statistical correlation with experimental results or numerical simulations so that resistance factors can be evaluated.

A.8.5.4 Bracing of beams, girders and trusses

A.8.5.4.1 Lateral bracing for beams and columns shall be capable of maintaining the laterally braced members in the laterally deflected position when the members are subject to their maximum factored forces. The stiffness of bracing members shall be sufficient to restrict the growth of out-of-plane deflections at the braced locations to a value consistent with that used in the analysis. The strength of bracing members shall be sufficient to maintain equilibrium.

In lieu of more detailed calculations the lateral bracing shall have a resistance of at least 1 % of the compressive force (in a column or in the compression flange of a beam) but not less than

$$F_{min} = 0.005 \frac{f_y}{\gamma_r} A_g$$

where A_g = gross area of the column or compression flange.

A.8.5.4.2 The initial out-of-straightness of a set of n columns or beams used to calculate the forces developed in the lateral bracing shall be taken as

$$\delta_n = (0.2 + \frac{0.8}{\sqrt{n}})\, \delta_1$$

where δ_1 = out-of-straightness for a single member.

For bracing a single member at midlength and growth of out-of-plane deflections is limited to the initial out-of-plane imperfection, the extensional stiffness of the bracing member shall in general not be less than 4N/L, where N is the compressive force in the member and L is the half-length of the member.

Where a line of bracing provides lateral support to a series of columns or beams the force-deformation characteristics of the bracing shall be taken into account.

A.8.6 Buckling of plates

A.8.6.1 General

This section covers the buckling resistance of rectangular thin plates with small initial geometric imperfections. The plates are assumed to be subjected to in-plane forces only, i.e. normal and shear stresses. The ultimate capacity will for slender plates occur after the plate has buckled and its deformations are much larger than the plate thickness, i.e. the plate has reached its post critical range.

With such deformations of the plate, the in-plane stresses will be concentrated towards the supported plate edges. The resistance of the plate may then be determined by introducing an effective width or an effective area of the plate.

Alternatively, the resistance may be calculated by using the average stress which the plate actually can sustain at its ultimate limit state.

The formulae given in A.8.6.2 - A.8.6.5 are empirical and are the results of extensive test programs. The formulae give the ultimate load which the plate can sustain, and there are no such unmobilized reserves in its capacity as may be obtained from a theory of small deformations.

A.8.6.2 Plates subjected to uniaxial force or in-plane moment

A.8.6.2.1 The resistance of the plate will be a function of the material strength f_y and the relative slenderness defined by

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}}; \quad \text{with} \quad \sigma_{cr} = k\sigma_E = k\,\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{B}\right)^2$$

$$\bar{\lambda}_p = 1.05\,\frac{b}{t}\,\frac{1}{\sqrt{k}}\sqrt{\frac{f_y}{E}}$$

The coefficient k depends on the stress distribution and the support conditions whether free, simply supported or restrained. k is given in Fig. A.8.6.2.1.a for the most common cases of axial force and moment.

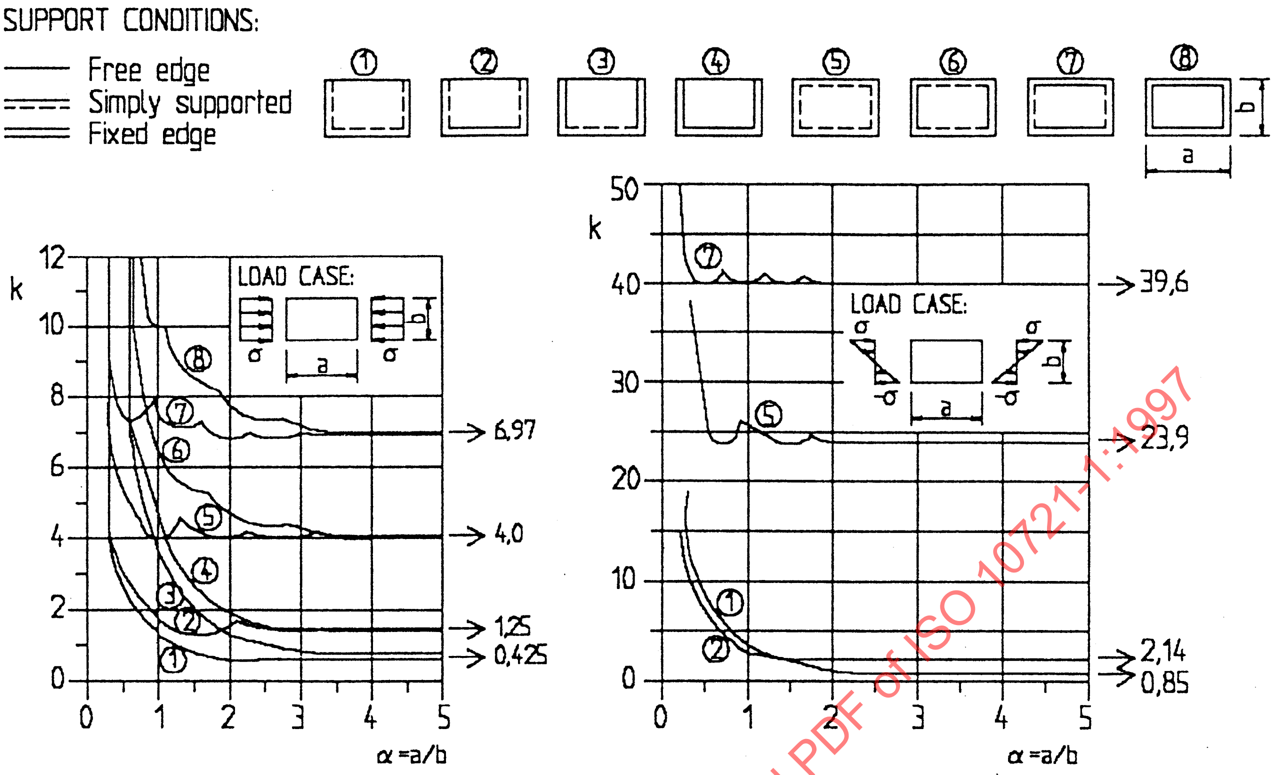


Fig. A.8.6.2.1.a Coefficient k

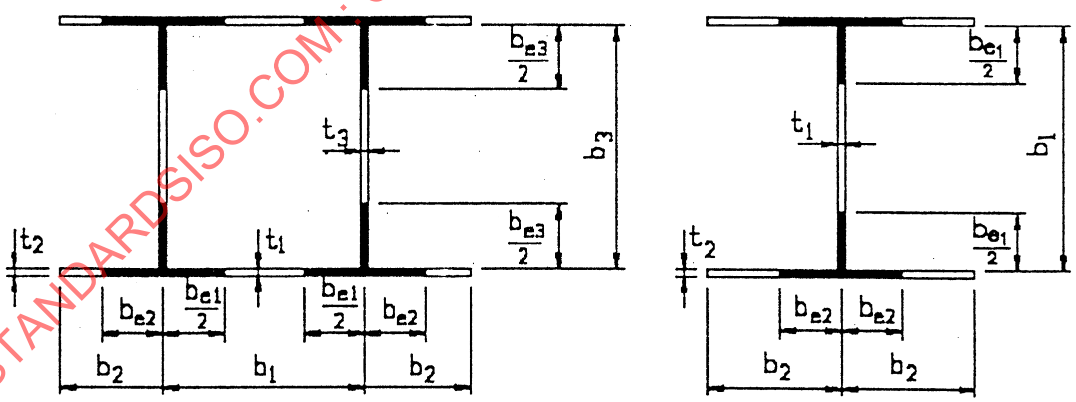


Fig. A.8.6.2.1.b

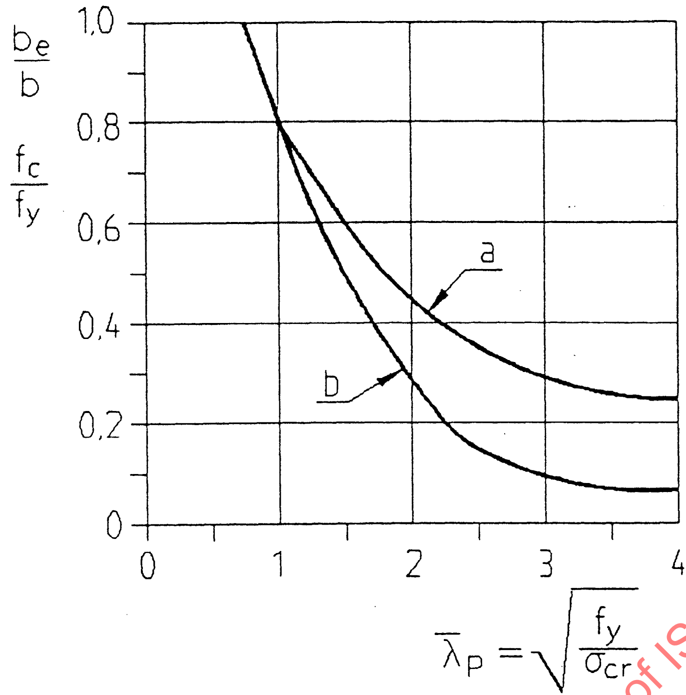


Fig. A.8.6.2.1.c

Table A.8.6.2.1 Effective width b_e and critical stress f_{cp} for fatigue loading for rectangular plates subject to uniaxial plane stress

Plate supported at 4 or 3 edges and prevented from lateral buckling	f_{cp} for plate subject to fatigue
$\frac{A_e}{A} = \frac{b_e}{b}\ 1$ when $\bar{\lambda}_p \le 0.71$ $\frac{A_e}{A} = \frac{b_e}{b} = \frac{1}{\bar{\lambda}_p}(1.00 - \frac{1}{4,9\bar{\lambda}_p})$ when $0.71 \le \bar{\lambda}_p \le 5.0$	$\frac{f_{cp}}{f_y} = 1$ when $\bar{\lambda}_p \le 0.71$ $\frac{f_{cp}}{f_y} = 1.5 - \frac{1}{\sqrt{2}}\bar{\lambda}_p$ $0.71 \le \bar{\lambda}_p \le \sqrt{2}$ $\frac{f_{cp}}{f_y} = \frac{1}{\bar{\lambda}_p^2}$ when $\sqrt{2} \le \bar{\lambda}_p \le 5$
$\frac{A_e}{A} = \frac{b_e}{b}$ is given as curve a in Fig. A.8.6.3.1.c	$\frac{f_{cp}}{f_y}$ is given as curve b in Fig. A.8.6.3.1.c

The edge of the plate shall be considered free or simply supported, unless it can be demonstrated that its edge is effectively restrained or fixed.

Plates subjected to repetitive actions should be checked with respect to possible fatigue effects (see Chapter 10) if the actions cause the plate to enter the post-critical buckling stage and "breathing" occurs. This may be avoided in the design by adopting the b-curve in Fig. A.8.6.2.1.c/Table A.8.6.2.1, which represents the linear elastic buckling curve. Its position may, however, be affected by geometrical imperfections.

In Fig. A.8.6.2.1.c and Table A.8.6.2.1 formulae are given for the effective width b_e and the critical stress f_{cp} for fatigue loading.

The load case may be axial force or bending moment. Combinations of axial force and moment are handled in A.8.6.4.

The effective area of a plate will be $A_e = t\,b_e$ concentrated along the supported edges as shown in Fig. A.8.6.2.1.b.

Plates supported at 4 edges, of which two are parallel to the stress direction will have two effective zones, one at each edge parallel to the stress direction.

Plates supported at 3 edges and with the fourth edge free and parallel to the stress direction, will have only one effective zone adjacent to its supported edge. Such plates must be prevented from lateral buckling.

If the section is part of a strut or column, the slenderness g of the column may be calculated for the gross-section.

The axial compressive (cross-sectional) resistance of a member is given by

$$N_{cd} = \frac{1}{\gamma_r} \cdot f_y \cdot A_e \quad \text{or} \quad N_{cd} = \frac{1}{\gamma_r} \cdot f_{cp} \cdot A$$

where f_{cp} is the lowest plate buckling strength.

A.8.6.2.2 Moment resistance of webs

The effective height h_e of a web in compression or bending is calculated according to the formulae given in Table A.8.6.2.1 or the curves of Fig. A.8.6.2.1.c, by replacing b_e/b with h_e/h_c, where h_c is the compressed part of the web calculated according to Navier's hypothesis, see Fig. A.8.6.2.2. The buckling coefficient k shall, however, be calculated for the entire web or for the appropriate parts of the web between longitudinal stiffeners.

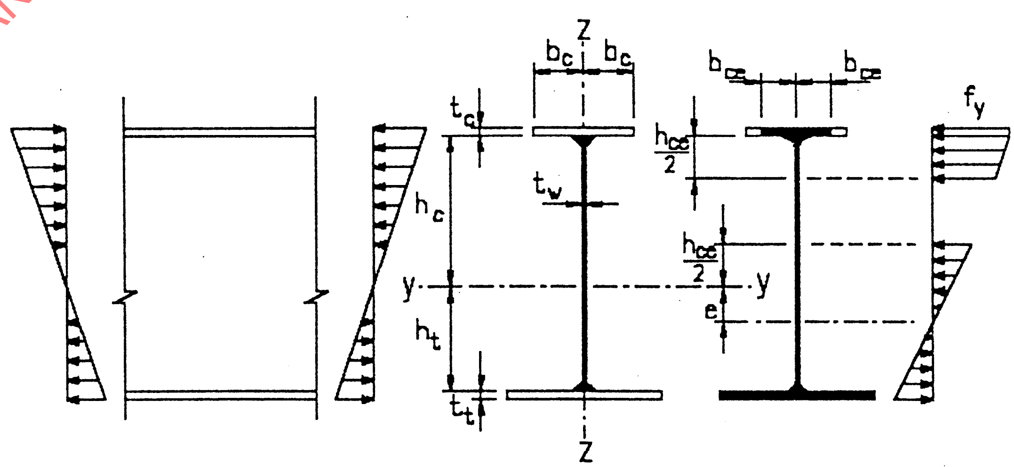


Fig. A.8.6.2.2

A.8.6.2.3 The minimum thickness of a web is given by

$$\frac{t_w}{h} \geq 2.4 \frac{f_y}{E}$$

which will prevent the compression flange of a straight beam to buckle into the web. h is the height of the web, or the distance from the flange to the nearest longitudinal stiffener. If the beam is curved in the web-plane, a more comprehensive investigation should be carried out.

A.8.6.2.4 For webs with axial force and non-uniform bending moment, the critical cross section to be checked is located either at 0.4a or at 0.4b from the end of the panel, see Fig. A.8.6.2.4.

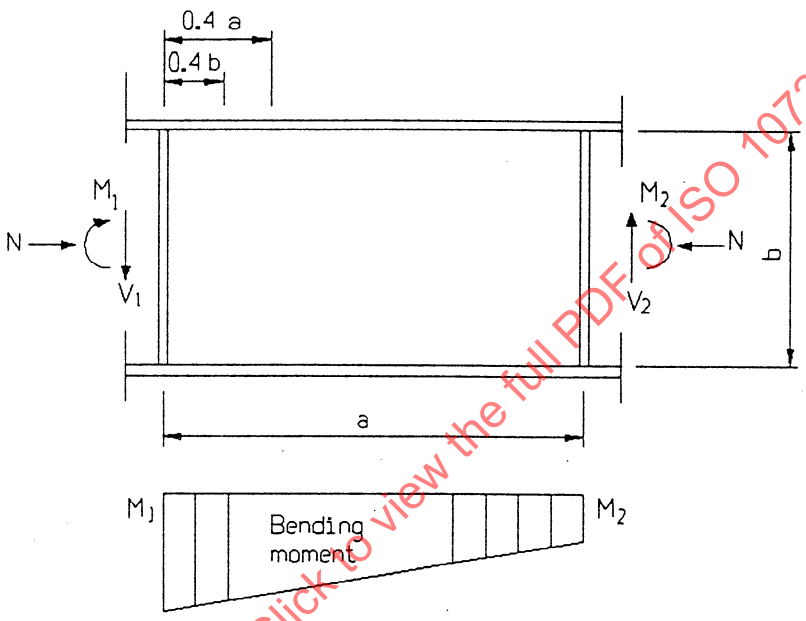


Fig. A.8.6.2.4 Control section of web.

A.8.6.3 Shear resistance of webs

A.8.6.3.1 The shear resistance of webs may either be calculated according to 8.6.3.2 and A.8.6.3.2 or according to a well-establiched tension field model.

Generally, the tension field model can be based on the assumption that the ultimate shear resistance is evaluated from a co-operative action between

- a simple shear field mechanism, and
- a tension field mechanism

The two mechanisms may be evaluated independently of each other, and the shear resistance of the panel is the sum of the shear forces which can be obtained for each of the two mechanisms. However, the calculations of the shear forces which can be obtained for each of the two mechanisms must be limited by their joint effect, as discribed below.

The shear force which can be obtained from the simple shear field mechanism (i.e. purely shear), should, under these assumptions, be limited by elastic shear buckling and by the shear yield stress of the web.

The shear force which can be obtained from the tension field mechanism should be limited by the yield strength of the web material. However, this mechanism may also include the effects of

- a frame mechanism,

in which case the shear force resistance will also be limited by the plastic resistance of the adjoining flanges and stiffeners.

The stress calculations in the web, and the plastic resistance calculations of the flanges and stiffeners must include all relevant effects from the co-operative mechanisms, and also the effects of other possible external forces (M and N).

Welds must be designed to resist all the relevant stresses which are introduced into them by the co-operative action of the mechanisms.

End panels must be given special attention, as the stiffness of the end stiffener will affect the tension field behaviour of the panel. The end stiffener must be designed to resist all forces which are introduced into it due to the co-operative actions of the mechanisms.

The recommendations given in A.8.6.4 and A.8.6.5 are applicable only if A.8.6.3.2 is applied to calculate the shear resistance of the web.

A.8.6.3.2 The method given below is an empirical method which gives the ultimate shear resistance of a web.

The ultimate shear resistance of webs may be expressed as a function of the material strength and the relative web slenderness, $\bar{\lambda}_p$,

$$\bar{\lambda}_p = \sqrt{\frac{\tau_y}{\tau_{cr}}} = 0.8 \cdot \frac{h}{t} \frac{1}{\sqrt{k_s}} \sqrt{\frac{f_y}{E}}$$

The support conditions shall always be assumed simply supported, and k_s is given by

$$k_s = 5.34 + 4\left(\frac{h}{a}\right)^2 \quad \text{for} \quad a \geq h$$

$$k_s = 5.34 + 4\left(\frac{h}{a}\right)^2 + 4.00 \quad \text{for} \quad a \leq h$$

where a is the distance between transverse stiffeners.

The resistance of a web also depends on the flexural stiffness in the web plane of its stiffeners. See 8.6.5 and A.8.6.5.

In table A.8.6.3.2 formulae are given for the ultimate shear strength of a web, τ_c, as a function of $\bar{\lambda}_p$ and the aspect ratio α = a/h. The shear yield stress

$$\tau_y = f_y / \sqrt{3}.$$

The formulae of Table A.8.6.3.2 are all empirical and based on extensive test results. Rigid end stiffeners are defined in A.8.6.5.5.

The ultimate shear resistance of the web is given by

$$V_{cd} = \frac{1}{\gamma_r} \cdot \tau_c \cdot t \cdot h$$

Table A.8.6.3.2. Ultimate shear strength

Web with rigid end stiffener	Web with flexible end stiffener
$\frac{\tau_c}{\tau_y} = 1$ for $\bar{\lambda}_p \le \frac{0.76}{1-0.024(4-\alpha)^2}$ $\frac{\tau_c}{\tau_y} = \frac{0.76}{\bar{\lambda}_p} + 0.024(4-\alpha)^2$ for $\frac{0.76}{1-0.0124(4-\alpha)^2} < \bar{\lambda}_p \le 1.22$ $\frac{\tau_c}{\tau_y} = \frac{1.52}{\bar{\lambda}_p+1.22} + 0.024(4-\alpha)^2$ for $1.22 < \bar{\lambda}_p \le 3.8$	$\frac{\tau_c}{\tau_y} = 1$ for $\bar{\lambda}_p \le 0.76$ $\frac{\tau_c}{\tau_y} = \frac{0.76}{\bar{\lambda}_p}$ for $0.76 < \bar{\lambda}_p \le 3.8$

For $\alpha > 4$, $\alpha = 4$ should be used in the formulas in Table A.8.6.3.2.

Webs subjected to repetitive shear action should be checked with respect to possible fatigue effects (see chapter 10) if the action cause the plate to enter the post-critical buckling stage and "breathing" occurs. This may be avoided in the design by adopting the following:

$$\frac{\tau_c}{\tau_y} = 1 \qquad \text{for} \qquad \bar{\lambda}_p \le 0.76$$

$$\frac{\tau_c}{\tau_y} = \frac{0.76}{\bar{\lambda}_p} \qquad \text{for} \qquad 0.76 < \bar{\lambda}_p \le 1.41$$

$$\frac{\tau_c}{\tau_y} = \frac{1.08}{\bar{\lambda}_p^2} \qquad \text{for} \qquad 1.41 < \bar{\lambda}_p \le 3.8$$

A.8.6.4 A combination of forces

A.8.6.4.1 A combination of axial force and moment may be handled by checking the sum of the uniaxial stresses against the appropriate buckling strengths according to A.8.6.2 for each part of the cross-section, in which case the elastic buckling coefficient k must be calculated for the combined uniaxial stress distribution.

If the member is an I-section subjected to a combination of axial force, strong axis moment and shear, the web of the cross-section may be checked by the interaction formula

$$\frac{N}{N_{rd}} + \left(\frac{M}{M_{rd}}\right)^2 + \left(\frac{V}{V_{rd}}\right)^2 \le 1$$

in which all terms exclusively refer to the web, and N_{rd}, M_{rd} and V_{rd} are calculated in accordance with A.8.6.2 and A.8.6.3.2, i.e

$$N_{rd} = \frac{f_y}{\gamma_r} A_e$$

$$M_{rd} = \frac{f_y}{\gamma_r} W_e$$

$$V_{rd} = \frac{\tau_c}{\gamma_r} A_w$$

If the panel have axial forces in two directions (x and y), the first term of this equation may be replaced by

$$\frac{N_y}{N_{yd}} + \frac{N_z}{N_{zd}}$$

A separate control of the flanges may be carried out by checking the stresses against the appropriate buckling strength given in A.8.6.2.

Alternatively, a tension field theory comprising the entire cross-section may be adopted.

For a member with an I-section subjected to a combination of strong axis moment and shear, the following empirical interaction formulae may be adopted

$$0.73 \frac{M}{M_{rd}} + 0.46 \frac{V}{V_{rd}} \leq 1$$

$$\frac{M}{M_{rd}} \leq 1 \quad \text{and} \quad \frac{V}{V_{rd}} \leq 1$$

in which all terms refer to the entire cross-section, and M_{rd} and V_{rd} are calculated in accordance with A.8.6.2 and A.8.6.3.2.

A.8.6.4.2 Local buckling or crippling of web

This problem concerns mainly crane girders, where heavy wheel loads may produce crippling and yielding in the web just beneath the wheel load. The result may be extensive damage to the upper part of the web. (See also chapter 10, Fatigue).

The crippling resistance of an unstiffened web subjected to a concentrated load P, as illustrated in Fig. A.8.6.4.2, may be calculated by

$$P_d = \frac{f_y}{\gamma_r} t_w a \qquad \text{for class 1 sections}$$

$$P_d = \frac{f_y}{\gamma_r} t_w^2 \sqrt{f_y / E} \qquad \text{for class 2, 3 or 4 sections}$$

where

$$a = a_1 + 2 a_2 + 5 (t_f + r)$$

if P is located at a distance less than h from the end of the unstiffened web.

$$a = a_1 + a_2 + t_f + r \leq 500\,t_w^2/h$$

if P is located at a distance less than h form the end of the unstiffened web.

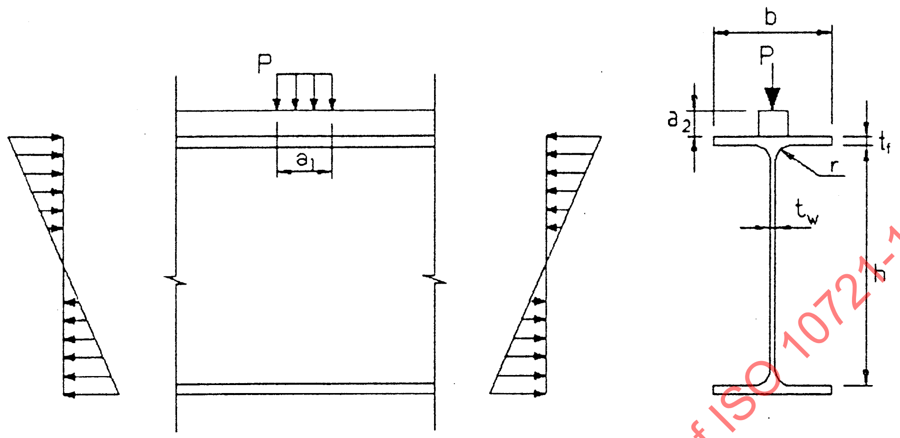


Fig. A.8.6.4.2

The web must also fulfil the requirements given in A.8.6.2.3.

A combined stress situation may be checked by simple theory of elasticity and the Huber-Hencky-von Mises yield criterion given in A.7.3.2. The local maximum stress in the web due to P (located farther than h from the end of the member) may be calculated by

$$\sigma_y = \frac{P}{t_w L_s}$$

where

$$L_s = 1.2 \left[a_1 + 2 \left(a_2 + t_w + r \right) \right] \qquad \text{or}$$

$$L_s = 1.4\, t_f \left(b/t_w \right)^{1/3}$$

whichever is the smallest, and b is the width of the flange, see Fig. A.8.6.4.2.

For slender webs loaded from both sides the local buckling resistance may also have to be checked. This may be done by considering the web as a fictitious column with a width equal to its height. If P is located nearer than h from the unstiffened end of a web, this width should be reduced accordingly. Buckling curve c may be applied.

A.8.6.5 Webs or panels subdivided by stiffeners

A.8.6.5.1 One or more stiffeners will increase the strength of the web or panel. The full effect will be obtained when the stiffeners behave as straight edges of the subdivided plate or web, and the ultimate plate resistance can be obtained before any significant buckling deformations of the stiffeners occur.

Fig. A.8.6.5.1.a shows some of the most commonly used open stiffeners.

Two fillet welds may give some restraining effect and accordingly improve the capacity.

Fig. A.8.6.5.1.b illustrates some of the more common torsional stiffeners. The effect of such stiffeners will be an effective restraint of the plate edges, and considerably increasing the resistance of the plate.

The moment of inertia for a stiffener is calculated with respect to the centre-line of the web, i.e. the z-axix as indicated in Fig. A.8.6.5.1.c.

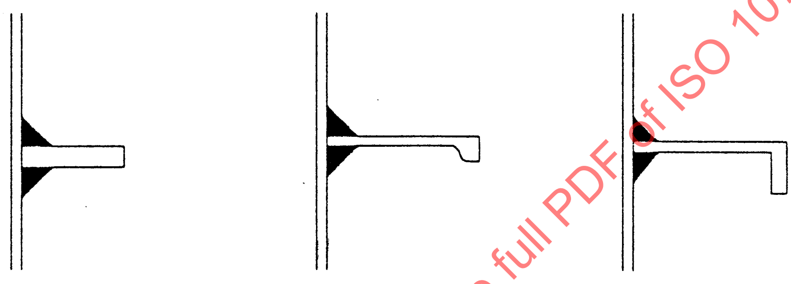


Fig. A.8.6.5.1.a Examples of open stiffeners

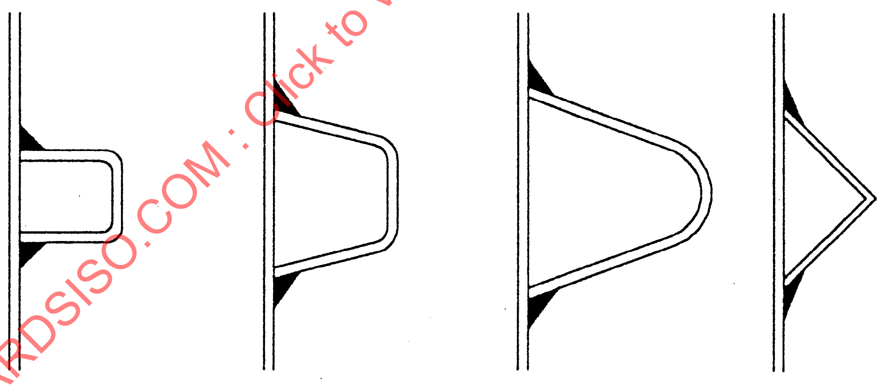


Fig. A.8.6.5.1.b Examples of closed stiffeners

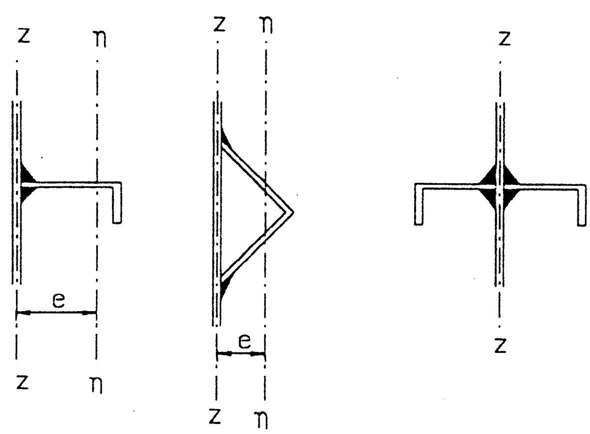


Fig. 8.6.5.1.c Types of stiffeners

A.8.6.5.2 For transverse stiffeners of plate girder webs, the moment of inertia I_s and the area A_s of the stiffener should fulfil the following requirements:

$$I_s \geq a\,t_w^3 \left[2.5 \left(\frac{h}{a}\right)^2 - 2 \right] \geq 0.5\,a\,t_w^2$$

$$A_s \geq 0.5\,a t_w \left(1 - \frac{\alpha}{\sqrt{1+\alpha^2}}\right) c_t\,\psi\,\varphi$$

where

$$c_t = 1 - \frac{1}{\bar{\lambda}_p^2} \geq 1.0$$

$$\phi = \begin{cases} 1{,}0 \text{ for double sided stiffeners} \\ 1{,}8 \text{ for single angle stiffeners} \\ 2{,}4 \text{ for single plate stiffeners} \end{cases}$$

ψ = ratio of yield strength of web plate to the yield strength of stiffener

α = aspect ratio = a/h

$\bar{\lambda}_p$ = the relative web slenderness (see A.8.6.3.2).

The length of the transverse stiffener is the distance h between the flanges.

In general any vertical stiffener may be cut off and unattached to the web for a distance less than or equal to $6t_w$, and if possible larger than $4t_w$. See Fig. A.8.6.5.5.

A.8.6.5.3 If the longitudinal stiffener is supposed to form a rigid support for the adjacent plates, it shall at least have a moment of inertia which fulfils the requirement

$$I_s \geq 0{,}1\,h\,t_w^3\,C_l$$

where

$$C_l = 8 + 60\,\frac{A_s}{h\,t_w}$$

The effective area A_{Le} of the longitudinal stiffener may be calculated as given in A.8.2 and A.8.6.2. The stiffener itself should, when possible, at least have width/thickness ratios as required for calss 2 sections. The stiffener with the adjoining parts of the web may be calculated as a column, as indicated in chapter 8.4. The buckling length of this column is the distance between the transverse stiffeners or between points where deflection is prevented, unless more exact calculations are made.

The axial load in the column will be the sum of the forces due to the stresses in the stiffener itself and the stresses in half the width of the adjoining panels.

Longitudinal continuous stiffeners may be calculated as centrally loaded. Non-continuous stiffeners, e.g. stiffeners which are terminated at each transverse stiffener, shall be calculated as excentrically loaded, the axial load acting in the middle plane of the web.

A.8.6.5.5 End stiffener

If the web is calculated by an post-critical method as given in A.8.6.3.2, the end stiffener should be flexurally rigid, and it should be designed for

- an axial force equal to the entire support force acting on an equivalent column length of 0.75 h, and

- a horizontal force (due to the tension field action), see Fig. A.8.6.5.5,

$$H_t = h\,t_w\,(\tau_c - 0.76\,\frac{\tau_y}{\bar{\lambda}_p})\,\sqrt{1+\alpha^2}$$

where

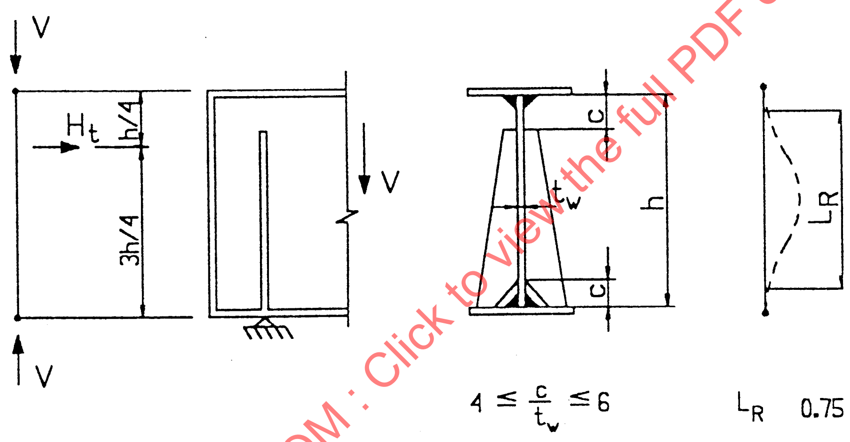
τ_c and $\bar{\lambda}_p$ are given in A.8.6.3.2, and H_t is located 0.25h from the top flange.

Alternatively, the end panel should be designed such that

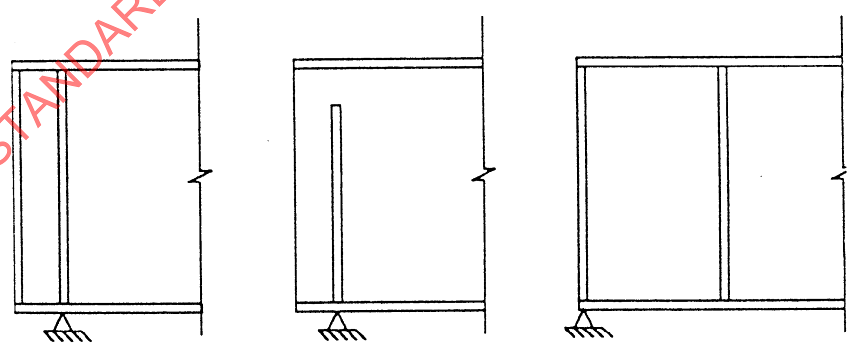
$$a \text{ or } h \le t\,[0.2V/(E\,A_w)]^{-0,5}$$

where

a is the distance between the end stiffener and the first transverse stiffener. This will ensure an adequate anchorage panel for the tension field.



a) Calculation model for end stiffeners



b) Examples of end panel stiffeners

Fig. A.8.6.5.5 End panel stiffeners

A.8.8 Bolted connections

A.8.8.2 Bolting details

A.8.8.2.1 Spacing of bolts

The bolt pitch (distance parallel to the direction of the action) shall preferably be three times the nominal diameter of the hole, but not less than 2.5 hole diameters. The pitch shall assure adequate strength and take fabrication and erection procedures into consideration. The maximum bolt pitch of the inner bolt rows parallell and transverse to the action of the force may be increased by 100 % in members with tension action.

The minimum edge distance at right angles to the direction of the action and measured from the centre-line of the hole to the edge of the plate shall not be less than 1.5 times the nominal diameter of the hole. The minimum edge distance may be reduced to 1.2 times the nominal diameter of the hole, provided appropriate strength calculations are made.

The minimum end distance in the direction of the action and measured from the centre-line of the bolt to the end of the plate or member shall not be less than 1.2 times the hole diameter for compression or tension members.

The maximum edge or end distance shall be the lesser of 12 times the thickness of the outside connected part or 150 mm, or as governed by local buckling or corrosion protection requirements.

In the case of unsymmetrical and unsymmetrically connected members (such as angles), appropriate considerations must be made to determine the strength of the connection, in addition to fulfilling the spacing requirements mentioned above.

A.8.8.2.2 Holes

For normal bolts there will be a clearance as the hole diameter will be up to 3 mm greater than the bolt diameter. The clearance is necessary for a convenient erection of the structure. The use of slotted or oversize holes are dictated by equivalent considerations.

Joints using oversize or slotted holes shall meet the following requirement:

a) Oversize holes shall not be used in bearing-type connections but may be used in any or all plies of slip-critical connections. Oversize holes shall not be more than 4 mm larger than the bolt diameter for bolt diameters up to 22 mm, not more than 6 mm larger than the bolt diameter for bolts of 24 mm in diameter, and not more than 8 mm larger than the bolt diameter for bolts of 27 mm or of larger diameters.

b) Short slotted holes are those 2 mm wider than the bolt diameter with a slot length not exceeding the oversize diameter provisions (8.8.2.2.a) by more than 2 mm. They may be used in any or all plies of slip-critical connections or in bearing-type connections. In slip-critical connections the slots may be orientated at any direction with respect to the direction of the actions. In bearing-type connections, the long direction of the slots shall be perpendicular to the direction of the action.

c) Long slotted holes are those 2 mm wider than the bolt diameters and with a length larger than prescribed for short slotted holes but not more than 2.5 times the bolt diameter. They may be used in only one ply at adjacent parts of a faying surface, in slip-critical or bearing-type connections. In slip-critical connections, they may be used without regard to the direction of the actions, but there shall be provided one-third additional bolts in excess of the number of bolts required to satisfy 8.8.3 and A.8.8.3. In bearing-type connections, the long direction of the slot shall be perpendicular to the direction of the action.

A.8.8.2.3 High-strength bolts

As a general rule a high-strength bolt subject to repetetive tensile loading should be preloaded. In connections where prying forces will occur, see Fig. A.8.8.2.3, this effect shall be included in the calculations.

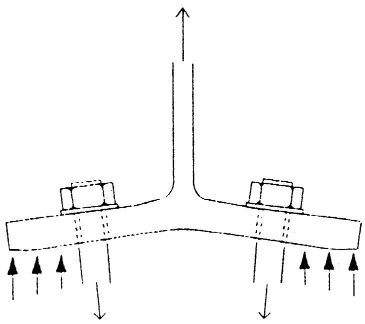


Fig. A.8.8.2.3

A.8.8.3 Strength of connections with bolts and rivets

A.8.8.3.1 Bolts in tension

The tensile force resistance of bolts is given by

$$F_t = f_d A_{sp}$$

where

$$f_d = \frac{f_u}{\gamma_{rc}}$$

A_{sp} = nominal stress area of the threaded part

f_u = specified ultimate tensile strength of the bolt material

γ_{rc} = resistance factor for the connection.

The effect of prying forces, if any, shall be included in the external tensile force.

A.8.8.3.2 Bolted joints subjected to shear forces.

A.8.8.3.2.1 Bolts in bearing and shear type connections

Bolts are subjected to shear by the bearing of the connected parts against the bolt shank. The resistance of the connection will therefore be governed either by the shear resistance of the bolts or by the bearing resistance of the connected parts.

a) The shear resistance of a bolt in a bearing-type connection is given by

$$F_v = \kappa \frac{f_u}{\gamma_{rc}} A_v$$

where:

κ = 0.6 for 8.8 and 10.9 bolts when shear occurs in the unthreaded area, 0.5 when shear occurs in the threaded area

A_v = effective shear area of bolt, including the effect of possible threads

f_u = specified ultimate tensile strength of bolt material

γ_{rc} = resistance factor for the connection

For connections with a spacing larger than 15 d between the first and last bolt, see A.8.8.6.

b) The bearing resistance per bolt, equal to or greater than the bearing strength of the plates adjacent to the bolt, is given by

$$F_b = \alpha f_u t d / \gamma_{rc}$$

where:

d = bolt diameter,

t = sum of plate thicknesses loaded in the same direction as the bearing stress,

and where α is the relation between the bearing strength and the ultimate strength of the member material; α is a function of the distance e from the center of the last bolt to the edge of the plate in the force direction, and a function of the distance between the bolts measured in the force direction (pitch).

For members subjected to tensile action

$$\alpha = 1.5 \cdot \frac{e}{d} \quad (e \geq 1.2\ d) \leq 3.0$$

For compression members: α = 3.0

c) Bolts in a bearing type connection may be subject to combined tension and shear. In this case, the following interaction equation may be applied:

$$\left(\frac{T}{F_t}\right)^2 + \left(\frac{V}{F_v}\right)^2 \leq 1$$

where:

T = tensile force in the bolt

F_t = tensile resistance of bolt, see A.8.8.3.1

V = shear force on the bolt for the shear plane

F_v = shear resistance of the bolt, see A.8.8.3.2 above, calculated for the shear plane.

A.8.8.3.2.2 Bolts in slip critical connections

The resistance to slip provided by a high-strength bolt may be taken as:

$$F_s = m \pi F_p \frac{1}{\gamma_{rs}}$$

where:

F_p = specified preloading force

γ_{rs} = slip resistance factor

m = number of faying surfaces

μ = slip coefficient, see A.8.8.4

If slip occurs the connection shall be considered as bearing-type connection, see A.8.8.3.2.1.

Bolts in a slip critical connection may be subjected to tension in addition to the shear. In this case, the slip resistance is given by:

$$F_s = m\,\mu\,(F_p - 0.8\,F_t)\,\frac{1}{\gamma_{rs}}$$

where:

F_t is the tensile force pr. bolt due to external actions and possible prying forces.

A.8.8.4 Slip coefficients

The slip coefficient strongly depends on the surface treatment. The effectiveness of spraymetalized surfaces depends on the spraying process, and the values of m controlled accordingly. The slip coefficient also depends on the type of action, and its variation with time. Representative short term values of m are given in Table A.8.8.4.

Table A.8.8.4. Short term values of the slip coefficient

Steel surface treatment	Slip coefficient (short term) μ
For weathered steel clear of all mill scale and with any loose rust removed	0.40
For surfaces blasted with shot or grit and with any loose rust removed	0.50
Tightly adhering clean mill scale, except for quenched and tempered steels	0.33
Hot dip zinc metalized	0.18
Hot dip zinc metalized and lightly blasted, thickness > 50 μmm	0.40
Spraymetalized with zinc, thickness > 50 μmm	0.40
Spraymetalized with aluminium, thickness > 50 μmm	0.55
Painted with zincsilicate coat, thickness < 60 μmm	0.50
Painted with zinc dust coat, thickness < 60 μmm	0.35

Alternatively, the slip coefficient for specific coating systems may be established by relevant tests or taken from national standards.

For structures in which slip into bearing under long term sustained loads would be detrimental, design recommendations should be found from long term tests or in the appropriate national standards. If slip will occur for such a connection, its resistance should be calculated according to the recommendations given for bearing type connections, see A.8.8.3.2.1.

A.8.8.6 Length of connection

When the spacing between the first and the last bolt in a connection is larger than 15d, where d is the bolt diameter, the resistance of the connection should be reduced by a factor β which is given in Fig. A.8.8.6.

However, this reduction does not apply for uniform distribution of forces over the connection length, e.g. for the transfer of shear forces from the web of a beam or column to the flange, nor does it apply to slip calculations of slip-critical connections.

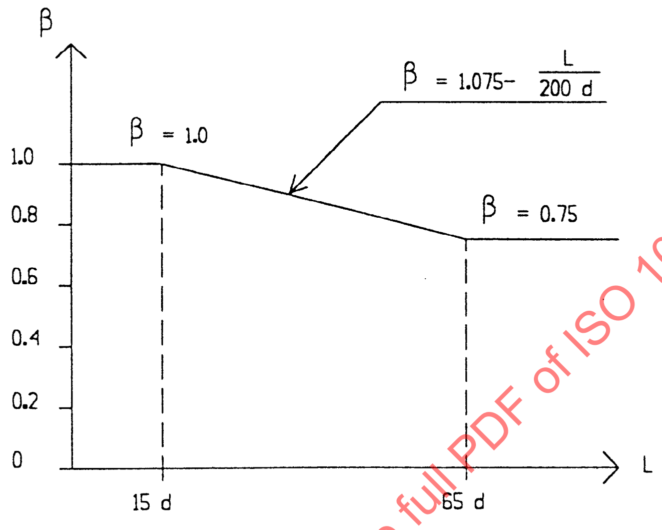


Fig. A.8.8.6 Reduction of capacity in long connections

A.8.9 Welded connections

A.8.9.2 General requirements

The weld material should match the material in the structure.

A.8.9.4 Design assumptions

For the calculation of weld sizes, a simplified stress distribution within the welds in the joint is normally assumed. Since the actual elastic distribution of forces between welds is highly indeterminate, such assumptions have been found acceptable and satisfactory for design practice, and rely on the demonstrated capacity of welds to redistribute stresses by plastic yielding. However, care must be taken in providing the necessary capability as well as the sufficient freedom of constraints in the configuration of the joints to permit such yielding and the resulting deformations to occur.

A.8.9.5 Design provisions

A.8.9.5.2 The resistance of a complete penetration groove weld in a butt joint should be the resistance of the weaker member.
Partial penetration groove welds should be calculated as fillet welds.

The resistance of a complete penetration groove weld in a Tee joint should be the resistance of the stem.

The resistance of a partial penetration groove weld should be calculated in the same manner as that for fillet welds in the same joint.

A.8.9.6 Groove welds in butt and tee joints

A.8.9.6.2 For T-joints with partial penetration groove welds, see Figs. A.8.9.6.2 and 8.9.7.10, the resistance may be calculated by handling the joint as one with fillet welds.

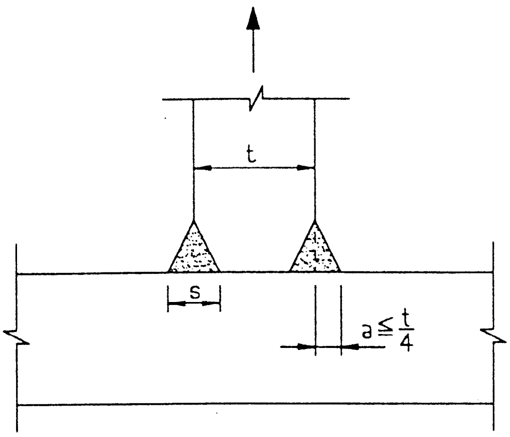


Fig. A.8.9.6.2

A.8.9.7 Fillet welds

A.8.9.7.1 The fillet weld may be controlled by calculation of forces in the different cross sections A, B and C, as illustrated in Fig. A.8.9.7.1, where the relevant stress components are shown for section A only.

The relevant stresses may be expressed by the following components:

$\sigma\parallel$ - normal stress parallel to the axis of the weld

$\sigma\perp$ - normal stress perpendicular to the plane of the weld throat containing the axis of the weld

$\tau\parallel$ - shear stress parallel to the axis of the weld

$\tau\perp$ - shear stress in the plane of the throat perpendicular to the axis of the weld

The normal stress component parallel to the axis of the weld, $\sigma\parallel$, may in resistance calculations of the weld be omitted, as it is accomodated by the base material.

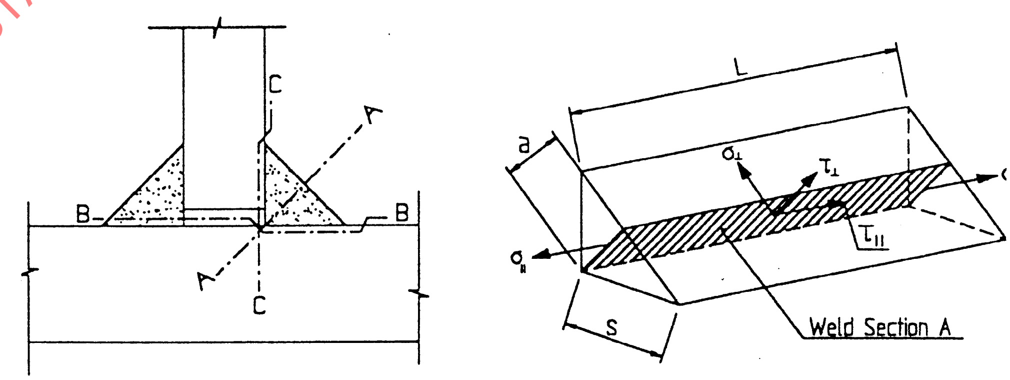


Fig. A.8.9.7.1

The resistance may be checked by

$$F \leq 0.6 \frac{f_u}{\beta \gamma_r} A_m$$

where

f_u = the ultimate tensile strength on the failure plane, i.e. on section A f_u is the tensile strength of the weld metal and on sections B and C f_u is the tensile strength of the base metal.

A_m = the appropriate effective area on section A, B or C

$$\beta = \begin{cases} 0.8 & \text{for } f_u = 360 \text{ N/mm}^2 \\ 0.85 & \text{for } f_u = 430 \text{ N/mm}^2 \\ 0.9 & \text{for } f_u = 510 \text{ N/mm}^2 \end{cases}$$

For section A the resistance may alternatively be checked by

$$\sqrt{\sigma_\perp^2 + 3\tau_\perp^2 + 3\tau_\parallel^2} \leq \frac{f_u}{\beta \gamma_r}$$

where

$\sigma\perp$, $\tau\perp$ and $\tau\parallel$ are the stress components on the effective area, due to the external design force F.

A.8.9.7.2 Single fillet welds are unable to take moments about their longitudinal axis.

A.10 FATIGUE

A.10.1 Scope

A.10.1.3 Situations in which no fatigue assessment is required

No fatigue assessment is required when one of the following conditions is satisfied:

- If, whatever the constructional detail, no nominal stress range multiplied by the factor γ_f exceeds $26/\gamma_r$ (N/mm²).

- If, for a particular detail, for which a constant amplitude limit is defined, no stress range (either nominal or geometric) multiplied by the factor γ_f exceeds the constant amplitude fatigue limit divided by the factor γ_r.

- If, whatever the detail, the number of stress cycles is less than:

$$2.10^6 \cdot \left(\frac{36}{\gamma_r \cdot \gamma_f \cdot \Delta\sigma_E}\right)^3$$

where

$\Delta\sigma_E$, the equivalent nominal stress range, is in N/mm².

A.10.2 Fatigue assessment procedures

A.10.2.1 Fatigue assessment based on nominal stress range

A.10.2.1.1 The number of cycles N_i to failure for a given stress range depends on the detail category, and may for the normal stress range $\Delta\sigma$ be calculated as:

- if $\gamma \cdot \Delta\sigma_i > \Delta\sigma_D$ then $N_i = 2 \cdot 10^6 \, (\Delta\sigma_c / \gamma \cdot \Delta\sigma_i)^3$
- if $\Delta\sigma_D > \gamma \cdot \Delta\sigma_i > \Delta\sigma_L$ then $N_i = 2 \cdot 10^6 \, (\Delta\sigma_c / \gamma \cdot \Delta\sigma_i)^6$
- if $\gamma \cdot \Delta\sigma_i < \Delta\sigma_L$ then $N_i = \infty$

with $\gamma = \gamma_f \cdot \gamma_r$

Alternatively, the fatigue assessment may be based on an equivalent constant amplitude stress range calculation, in accordance with the Palmgren-Miner rule of cumulative damage.

For normal stresses the fatigue assessment may thus be expressed by

$$\gamma_f \cdot \Delta\sigma_E \leq \Delta\sigma_R / \gamma_r$$

where

$\Delta\sigma_E$ is the equivalent constant amplitude stress range.

$\Delta\sigma_R$ is the fatigue strength, which depends on the detail category and the total number of stress cycles during the required design life.

A conservative assumption may be adopted in evaluating $\Delta\sigma_E$ and $\Delta\sigma_R$ in using a fatigue strength curve of unique slope constant m = 3.

More generally, $\Delta\sigma_E$ and $\Delta\sigma_R$ may be calculated taking into account the double sloped fatigue strength curve and the cut-off limit, se Fig. 3.1 at end of chapter 3.1.

A.10.2.2 Fatigue assessment based on a geometric stress range

The geometric stress (or the stress concentration factor applied on the nominal stress range) may be determined from parametric formulae within their domains of validity, a finite element analyses or an experimental model. The fatigue assessment based on geometric stress range is to be handled similarly to the procedures given in 10.2.1 replacing, where appropriate, the nominal stress range by the geometric stress range.

Reference fatigue strength curves to be used jointly with the geometric stress range concept are defined in A.10.5.2.

A.10.3 Fatigue loading

The fatigue loading may comprise different loading events which are defined by complete loading sequences of the structure, each characterized by their relative frequency of occurrence as well as their magnitude and geometrical position.

In the absence of more accurate information, dynamic amplification factors may be used to modify the stresses obtained from a static analysis.

The effect of a loading event is best described by its stress history which is the stress variation at a given point in the structure during the loading event.

Measured stress histories may not accurately reflect the future fatigue loading. In some structures, for example bridges and cranes, the load model used to describe the fatigue loading should, as such, be able to take into account the possible changes in usage, such as the growth of traffic, changes in the loading rate, etc.

Simplified design calculations may be based on an equivalent fatigue loading, representing the fatigue effects of all loading events. The equivalent fatigue loading may vary with the dimension and location of the structural element.

A.10.4 Fatigue stress spectra

A.10.4.2 Design stress range spectrum

The design stress range spectrum for a typical detail or structural element may be derived from the stress history obtained by adequate experiments and/or by numerical evaluations according to the theory of elasticity.

The Rainflow or the Reservoir stress cycle counting method, in conjunction with the Palmgren-Miner summation, is appropriate for many applications. The reservoir stress cycle counting method is illustrated in Fig. A.10.4.2.

A.10.5 Fatigue strength

The fatigue strength curves to be used in the fatigue assessment procedures are given according to the following classification of structural details:

- Nominal stress range procedures for
  * classified details for non-hollow sections
  * classified details for hollow sections, and hollow section joints in lattice girders.

- Geometric stress range procedure.

For the constructional details listed in Tables A.10.5.1.1.c to g, A.10.5.1.2.d and e the classification has been established on the basis of stresses along the direction indicated by the arrow for potential cracks on the surface of the parent metal, or for the case of weld throat cracking, on the stress calculated in the weld throat. The stresses are obtained from classical strength of materials elastic theory using the gross or net section of the loaded member, as appropriate. The stress thus calculated corresponds to details tested under simple loading configurations giving rise to a principal stress, generally parallel or almost parallel to the direction of the arrow used in the classification of constructional details, adjacent to the potential crack location. Note that the crack is located in a plane normal to this stress range direction. For these details the calculated stress is called the nominal stress, and the associated stress range for fatigue assessment, the nominal stress range.

The fabrication requirements for fatigue detail classifications are given under the heading "Requirements" of Table A.10.5.1.1 and in ISO Standard on "Fabrication and Erection". If the requirement for the actual details are not met in the finished structure, the use of the fatigue strength curves associated with those details may be inappropriate, in which case, a fatigue assessment must be carried out by suitable adaptation of these rules.

Test data for some details do not fit the fatigue strength curves given in Figure A.10.5.1.1.a. In order to be sure to avoid any non-conservative conditions, such details (identified by an asterisk in the tables) are located in a detail category one step lower than their fatigue strength at $2 \cdot 10^6$ cycles would have required. An alternative assessment would be to increase the classification of such details by one detail category provided that the constant amplitude fatigue limit is defined as the fatigue strength at 10^7 cycles for m = 3, se Fig. A.10.5.a.

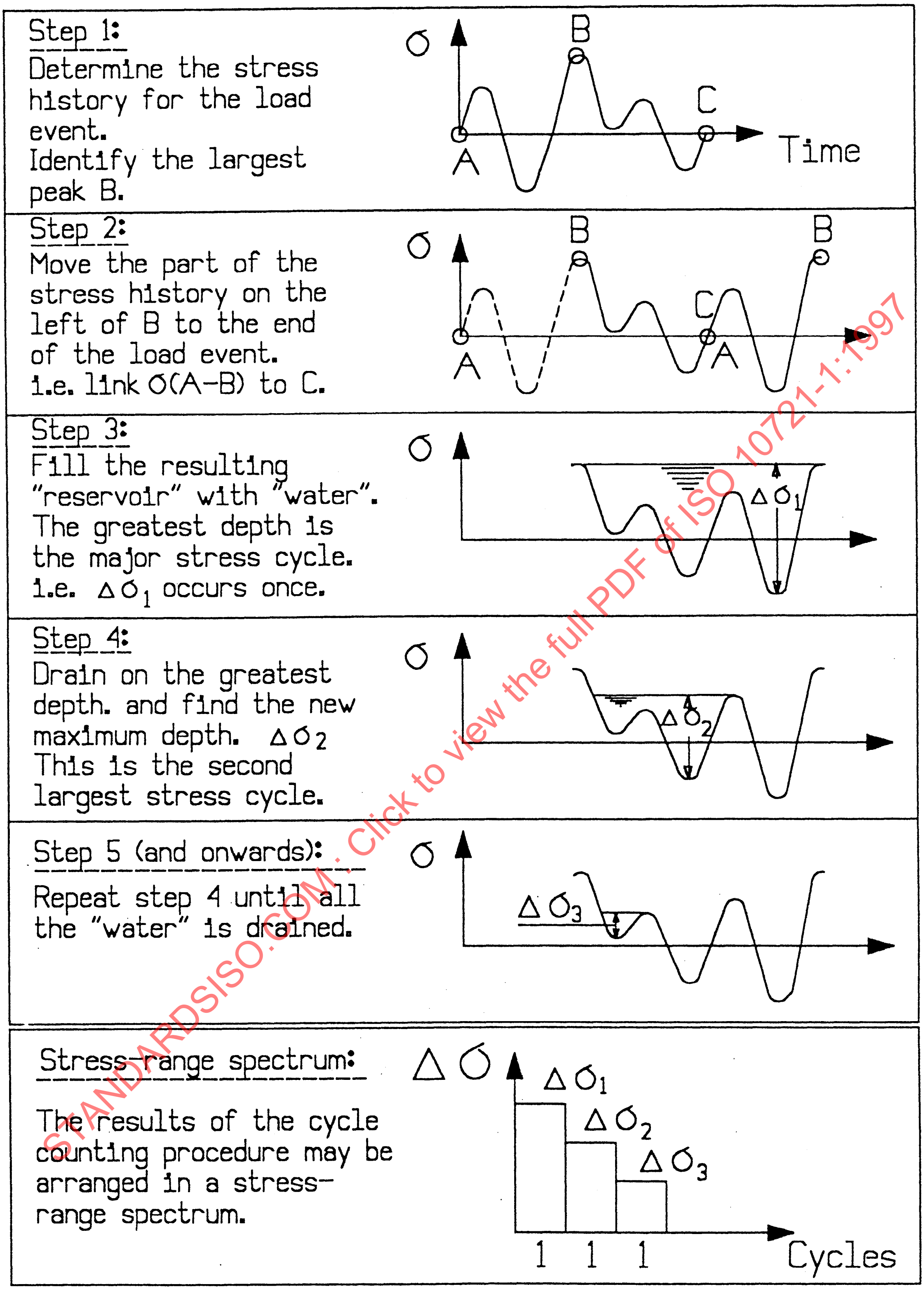


Fig. A.10.4.2 The reservoir stress cycle counting method

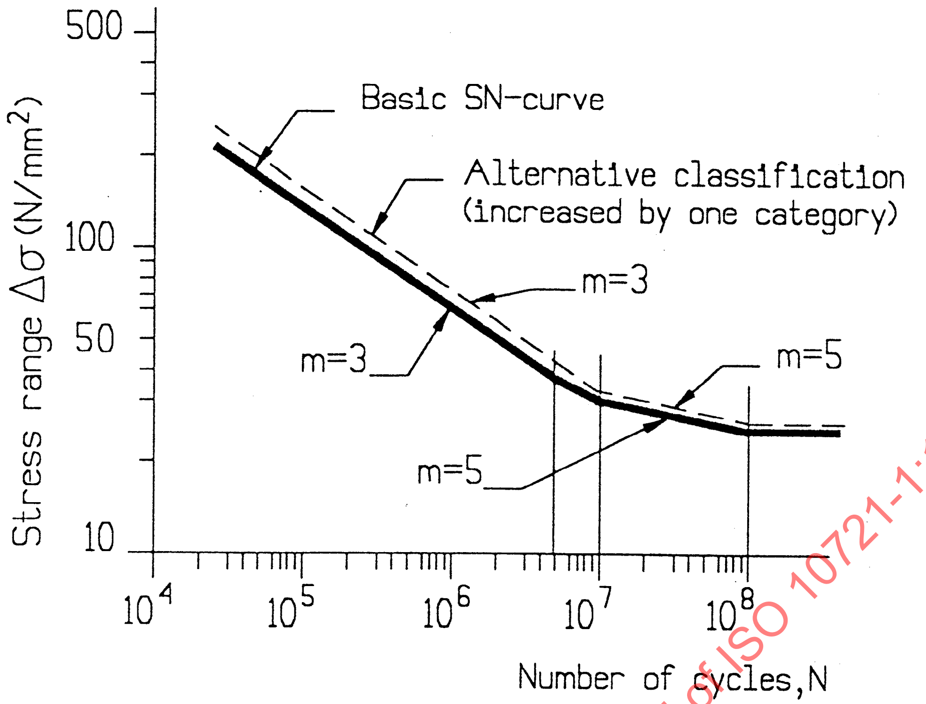


Fig. A.10.5.a

The requirements for the inspection of welded joints subject to fatigue should be in accordance with national standards, or the ISO Standard on Fabrication and Erection. Depending on the inspection scheme adopted, it may be necessary to identify certain critical joints on the design drawings and to elaborate on the procedures required for these joints.

In order that the appropriate degree of inspection may be applied to the various parts of the structure in accordance with the ISO "Fabrication and Erection" specification Appendix D, it is necessary to identify all joints where the stress spectrum is such that a detail category greater than 56 is required by these rules. In each case the joint should be identified on the detail drawing with the required "Fat" inspection category and stress direction, as illustrated on Fig. A.10.5.b. The required "Fat" inspection category number is the reference strength $\Delta\sigma_c$ for the lowest fatigue strength curve for which the damage summation is less than 1,0 (see 10.2.1.1).

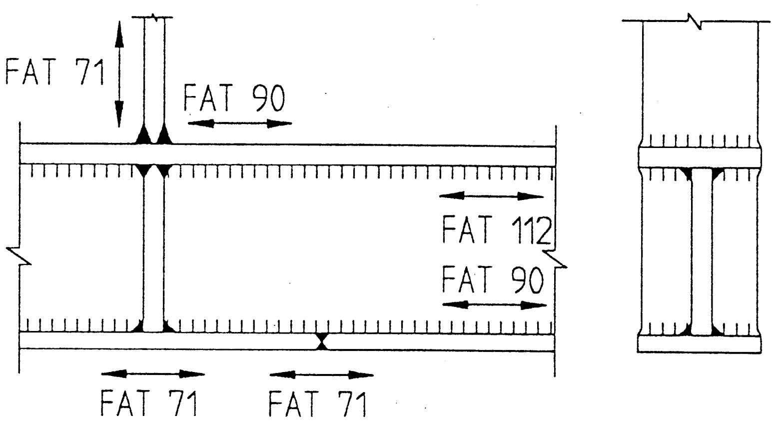


Fig. A.10.5.b

It is important that the value of the reference strength for the maximum permitted detail category according to the classification tables in A.10.5.1 is not used for the "Fat" number (unless it happens to be the same as the required "Fat" inspection category), otherwise unnecessary fabrication and inspection costs are likely to arise. Two or three "Fat" inspection categories may need to be called up at a joint if cycles stressing is severe in more than one direction.

A.10.5.1 Definition of fatigue strength curves for classified constructional details

A.10.5.1.1 Fatigue strength curves for non-hollow sections are defined as follows:

Classification of detail categories:

a) Typical constructional details for non-hollow sections are classified into 5 categories considering particularities in geometry and fabrication procedures:
- Non-welded details
- Welded built-up sections
- Transverse butt welds
- Welded attachments (non-load-carrying welds)
- Welded joints (load-carrying welds)

b) Fatigue strength curves for nominal normal stress range:

The fatigue strength curves for a number of typical detail categories are given in Figure A.10.5.1.1.a for nominal normal stress range.

The constant amplitude fatigue limit corresponds to the fatigue strength for $N = 5 \cdot 10^6$.

The cut-off limit corresponds to the fatigue strength for $N = 10^8$. Stress ranges below the value corresponding to the cut-off may be neglected.

The associated classification of various typical detail categories is given in the Tables A.10.5.1.1.c to g. The arrow in the different figures in the tables indicates the location and direction of the stresses for which the stress ranges are to be calculated.

The corresponding values for a numerical representation of the curves are presented in Table A.10.5.1.1.a.

c) Fatigue strength curves for nominal shear stress ranges:

The fatigue strength curves for nominal shear stress ranges are defined in Figure A.10.5.1.1.b and have a single slope constant of $m = 5$. Category 100 is for parent material, full penetration butt welds and for bolts of bearing type in shear. Category 80 is for fillet welds and for partial penetration butt welds in shear.

Calculations should be performed in a similar manner to those applied for nominal normal stress ranges. The cut-off limit remains at 10^8 cycles. No constant amplitude fatigue limit should be assumed.

The corresponding values for a numerical calculation of the fatigue strength are given in Table A.10.5.1.1.b.

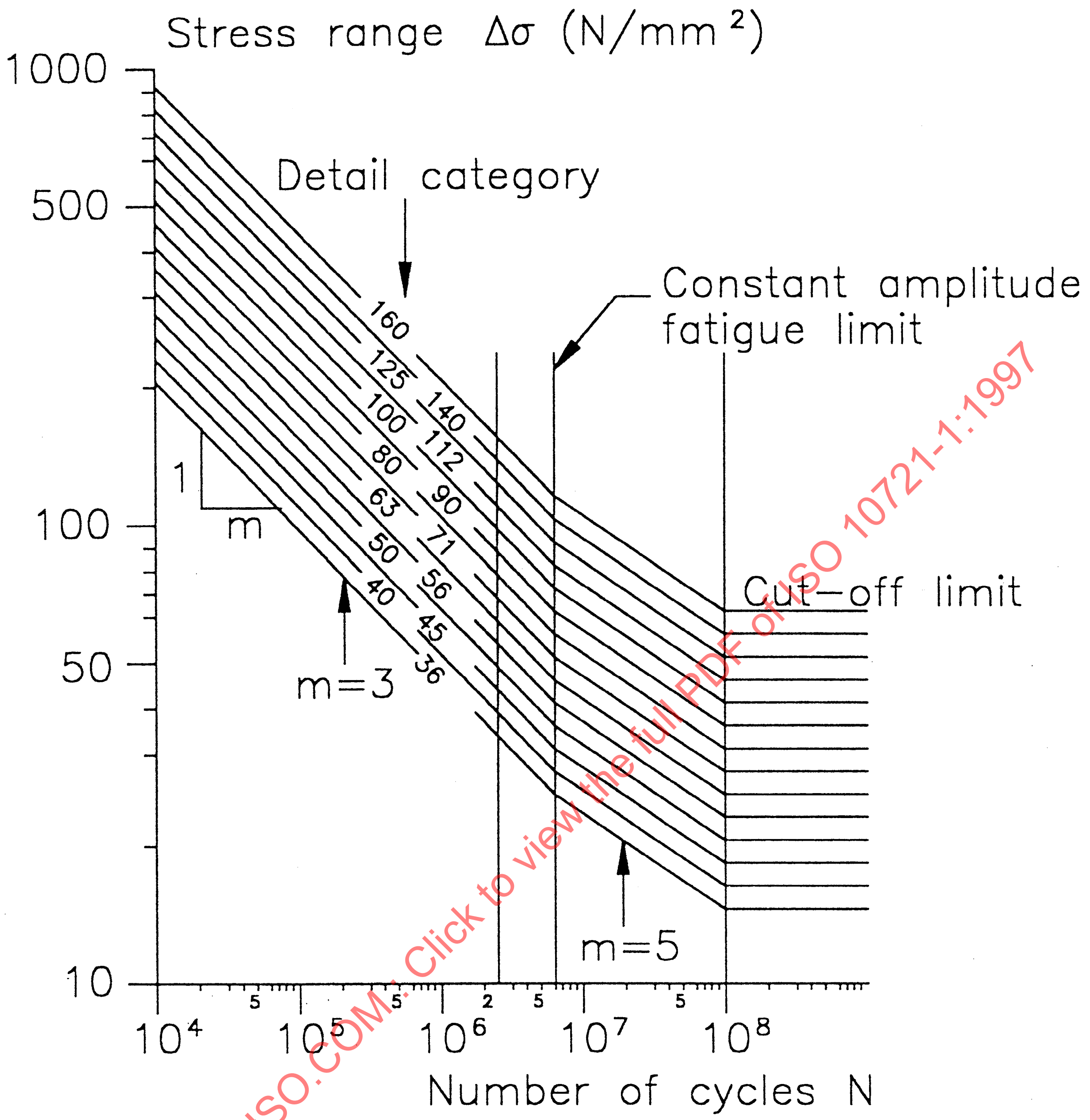


Figure A.10.5.1.1.a Fatigue strength curves for normal stress ranges

Table A.10.5.1.1 Numerical values of fatigue strength curves for normal stress ranges

Detail category $\Delta\sigma_c$ (N/mm²)	log a		Constant amplitude fatigue limit ($N = 5 \cdot 10^6$) $\Delta\sigma_D$ (N/mm²)	Cut-off limit ($N = 10^8$) $\Delta\sigma_L$ (N/mm²)
	($N \leq 5 \cdot 10^6$) m = 3	$N \geq 5 \cdot 10^6$) m = 5		
160	12.901	17.036	117	64
140	12.751	16.786	104	57
125	12.601	16.536	93	51
112	12.451	16.286	83	45
100	12.301	16.036	74	40
90	12.151	15.786	66	36
80	12.001	15.536	59	32
71	11.851	15.286	52	29
63	11.701	15.036	46	26
56	11.551	14.786	41	23
50	11.401	14.536	37	20
45	11.251	14.286	33	18
40	11.101	14.036	39	16
36	10.951	13.786	26	14

Table A.10.5.1.1.b Numerical values for fatigue strength curves for shear stress ranges

Detail category $\Delta\tau_C$ (N/mm²)	log a m = 5 ($N < 10^8$)	Cut-off limit ($N < 10^8$) $\Delta\tau_L$ (N/mm²)
100	16.301	46
80	15.801	36

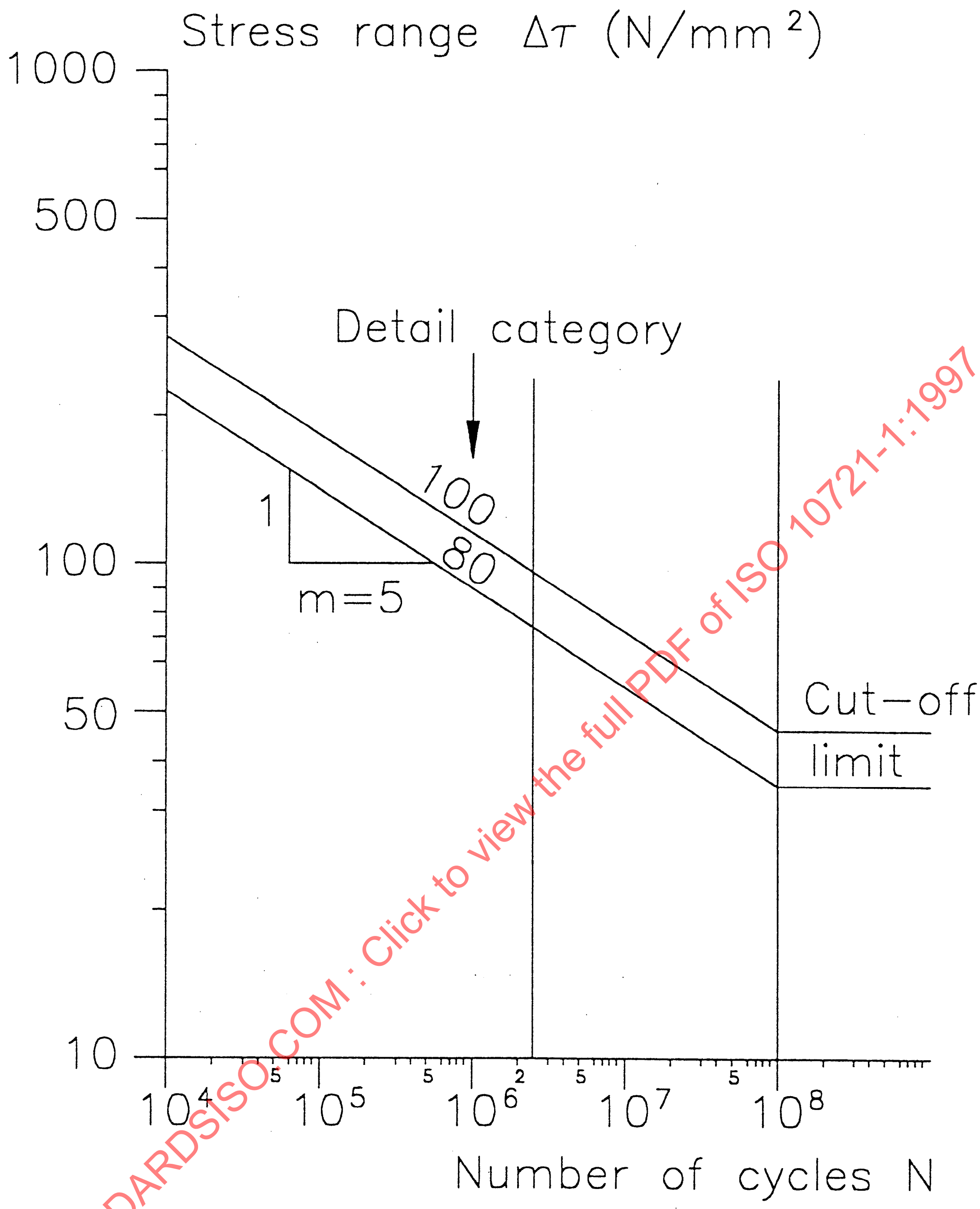
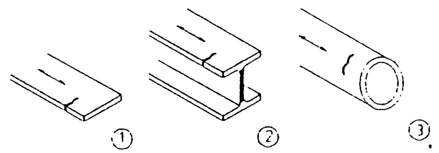
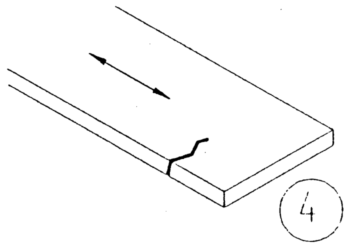


Figure A.10.5.1.1.b Fatigue strength curve for shear stress ranges

TABLE A.10.5.1.1.c: NON WELDED DETAILS

Detail category	Constructional Details	Description	Requirement
160		<u>Rolled and extruded products</u> 1 Plates, flates. 2 Rolled sections. 3 Seamless tubes (see Tables A.10.5.1.2 d-e).	1 to 3 : Sharp edges, surface and rolling flaws to be improved by grinding.
140		<u>Sheared or gas cut plates</u> 4 Machine gas cut or sheared material with no drag line. 5 Material with machine gas cut edges having shallow and regular drag lines or manual gas cut material	4 All visible signs of edges discontinuities should be removed. 5 Subsequently dressed to remove all edge discontinuities. 4 and 5 - no repair by weld refill. - Re-entrant corners (slope <1:4) or aperture should be improved by grinding for any visible defects. - In case of aperture, the design stress area should be taken as the net cross section area.
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