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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 3989:1987)

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_i , or it is the combination of nominal actions, each multiplied by its partial safety factor γ_i 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.
Repetitive 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 fatigue 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 structural 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 occurrence.
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 ($\Delta\sigma = \sigma_{\max} - \sigma_{\min}$ 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 literature 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).

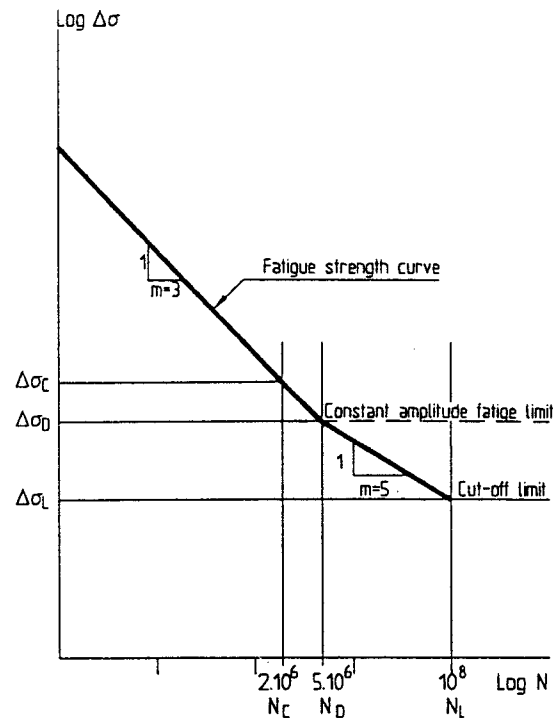


Fig. 3.1 Fatigue strength curve definitions

3.2

List of symbols

(see also ISO 3898)

LATIN UPPER CASE LETTERS:

A	Cross-sectional area
A_e	Effective cross-section area
A_o	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 = $E / (2(1 + \nu))$
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 = $\frac{\pi^2 E I}{L^2}$ (about the y- and z-axis, respectively).
N_{Ecr}	Elastic buckling load of a structure = $\frac{\pi^2 E I}{L_k^2}$
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 height
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 arbitrary 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_t	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
$\bar{\lambda}_0$	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 repetitive 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

6.1 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 action effects 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 1: 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 1 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 1 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

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

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_E 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 elastic buckling load.

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

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

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 λ , the yield strength f_y and the cross-sectional properties.

For cross-sections of class 1, 2 or 3, see 8.3.3, the resistance is

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

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 .

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

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

for class 1 and 2 sections

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

for class 3 sections, and by

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

for class 4 sections, unless a more detailed analysis is made, where f_{dL} 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_{dL}

The buckling strengths f_{cT} and f_{dL} 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 to 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, multiplied 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

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

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

$$\lambda_p = \sqrt{\frac{\tau_y}{\tau_c}} = 0.8 \cdot \frac{h}{t_w} \cdot \frac{1}{\sqrt{k_s}} \cdot \sqrt{\frac{f_y}{E}}$$

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

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

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.

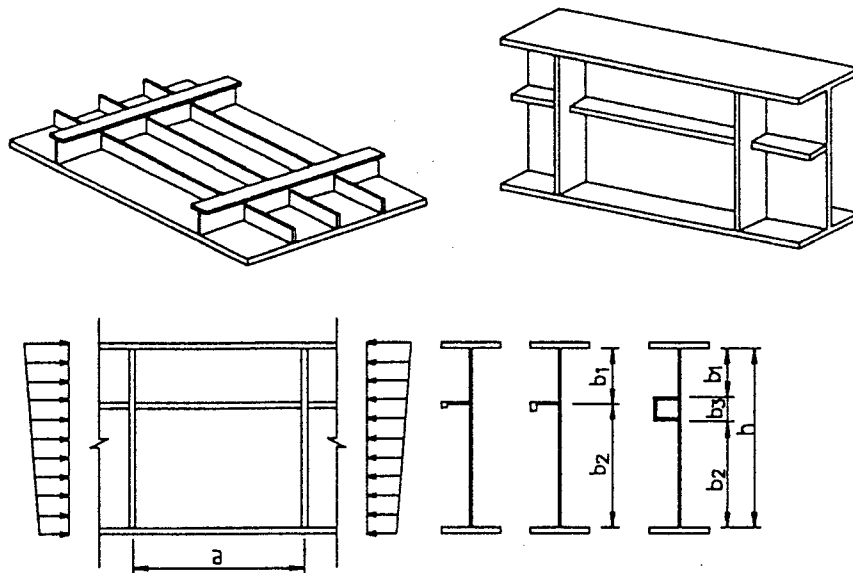


Fig. 8.6.5 Examples of transverse and longitudinal stiffeners

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 stiffeners 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 extending 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 $15d$ 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 $15d$, 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.

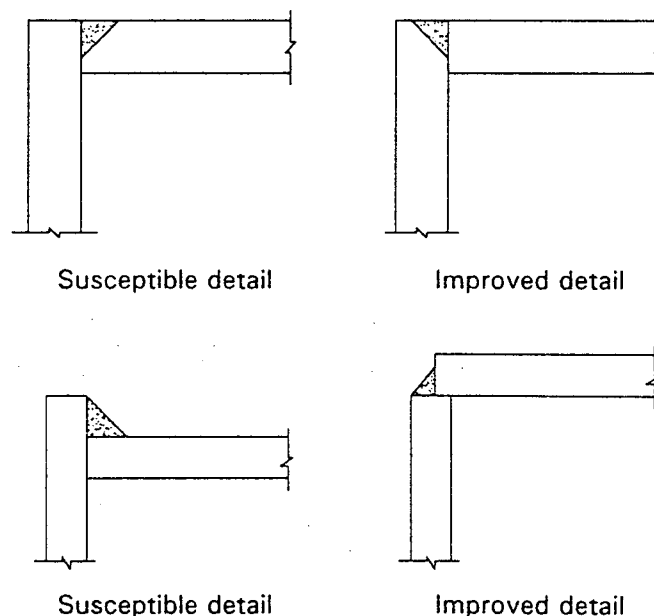


Fig. 8.9.2.7

- 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 provisions 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.