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Edition 2.1

SOUTH AFRICAN NATIONAL STANDARD

The structural use of steel

Part 1: Limit-states design of hot-rolled steelwork

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Table of changes

Change No.	Date	Scope
Amdt 1	2011	Amended to update referenced standards.

Foreword

This South African standard was approved by National Committee SABS SC 59F, *Construction standards – Steel and aluminium structures*, in accordance with procedures of the SABS Standards Division, in compliance with annex 3 of the WTO/TBT agreement.

This document was published in May 2011.

This document supersedes SANS 10162-1:2005 (edition 2).

A vertical line in the margin shows where the text has been technically modified by amendment No. 1.

SANS 10162 consists of the following parts, under the general title *The structural use of steel*:

Part 1: Limit-states design of hot-rolled steelwork.
Part 2: Limit-states design of cold-formed steelwork.
Part 4: The design of cold-formed stainless steel structural members.

Annexes A to H are for information only.

Introduction

This is the second edition in South Africa of a general limit-states design standard for steel structures. When the limit-states design standard was first introduced, it was designated Part 1 to distinguish it from cold-formed design (Part 2) and allowable stress design (Part 3). The allowable stress design standard has subsequently been withdrawn.

This standard is appropriate for the design of a broad range of structures. It sets out minimum requirements and is expected to be used only by engineers competent in this field. The scope recognizes that the requirements for the design of specific structures, such as bridges, are given in other international standards and that supplementary requirements may be needed for some particular structures.

Although the basic limit-states format as set out in the first edition has proven itself in use and remains unaltered, a number of technical changes reflecting the latest research developments and changes in practice have been incorporated. These changes are based on an increased understanding of the behaviour of structural materials and members, and thus of the overall behaviour of structures, as well as on major advances in methods of structural analysis and on improvements in fabrication and erection. Limit-states design has enhanced this development because the designer explicitly recognizes the different modes of failure and designs against these failure modes.

Introduction *(concluded)*

The clauses of this standard have been re-ordered and in some cases, combined. The annexes have been re-ordered: some have been brought into the standard, others have been eliminated. In addition, a new annex on crane-supporting structures has been added.

Specific changes in this standard include the following:

- semi-rigid construction is introduced as one of three types of construction;
- notional lateral loads are applied in all lateral load combinations and not as a minimum;
- the simplified method for bracing design is included as an alternative;
- high-strength bolts are assigned a higher resistance factor;
- web-bearing rules are simplified;
- a distinction is made between braced and unbraced frames;
- class 2 sections are reunited with class 1 sections in combined compression and bending;
- new clauses are provided for tension-shear block failures, for trusses (including a simple approach), and for composite columns;
- the clauses on bolting, welding and fatigue have been revised substantially.

The standard refers to SANS 2001:CS1, which defines the standards according to which steel structures have to be fabricated and erected. This standard does not apply to structures not meeting these standards as a minimum.

The standard is compatible with the load and combination factors defined in the national loading standard, SANS 10160 (all parts). **Amdt 1**

This standard is based largely on the Canadian standard, CSA S16, *Limit-states design of steel structures*. The assistance of the CISC and CSA is gratefully acknowledged. This South African standard has been prepared by a Technical Committee convened by the South African Institute of Steel Construction.

NOTE 1 Use of singular does not exclude the plural (and vice versa) when the sense allows.

NOTE 2 Although the intended primary use of this standard is stated in its scope, it is important to note that it remains the responsibility of the users of the standard to judge its suitability for their particular purpose.

NOTE 3 This publication was developed by consensus, which is defined as "substantial agreement". Consensus implies much more than a simple majority, but not necessarily unanimity.

NOTE 4 South African standards are subject to periodic review, and suggestions for their improvement will be referred to the appropriate committee.

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The structural use of steel

Part 1:
Limit-states design of hot-rolled steelwork

1 Scope

1.1 This standard provides rules and requirements for the design, fabrication and erection of steel structures. The design is based on limit states. The term "steelwork" refers to structural members and frames that consist primarily of hot-rolled structural steel components, and includes the detail parts, welds, bolts, fasteners and other items required in fabrication and erection. This standard also applies to structural steel components in structures framed in other materials.

1.2 Requirements for cold-formed steel structural members are given in SANS 10162-2.

1.3 This standard applies unconditionally to steelwork in buildings and other stationary structures, excluding road and rail bridges, antenna towers and offshore structures, except that supplementary rules or requirements may be necessary for:

a) unusual types of construction;

b) mixed systems of construction;

c) any steel structure that

   i) has great height or spans,

   ii) is required to be movable or readily dismantled,

   iii) is exposed to severe environmental conditions or possible severe loads, for example those resulting from vehicle impact or chemical explosion,

   iv) is required to satisfy aesthetic, architectural or other requirements of a non-structural nature,

   v) employs materials or products not listed in clause 5, or

   vi) has other special features that could affect design, fabrication or erection,

   vii) responds dynamically to loading, and

   viii) is designed to resist earthquakes;

d) tanks, stacks, other platework structures, poles and piling; and

e) crane supporting structures.

1.4 Alternative methods of design may be used in lieu of the formulae provided in this standard, provided that they comply with SANS 10160 (all parts) and provide nominal margins (or factors) of safety at least equal to those intended in the provisions of this standard (see annex B).

Amdt 1

2 Normative references

The following documents contain provisions which, through reference in this text, constitute provisions of this standard. All standards are subject to revision and, since any reference to a standard is deemed to be a reference to the latest edition of that standard, parties to agreements based on this standard are encouraged to take steps to ensure the use of the most recent editions of the standards indicated below. Information on currently valid national and international standards may be obtained from the SABS Standards Division.

2.1 Standards

AWS D 1.1, *Structural welding code – Steel*.

CSA B95, *Surface texture (Roughness, waviness and lay).*

CSA S16, *Limit states design of steel structures.*

EN 10025, *Hot-rolled products of non alloy structural steels: technical delivery conditions.*

EN 10155, *Structural steels with improved atmospheric corrosion resistance: technical delivery conditions.*

ISO 657-14, *Hot-rolled steel sections – Part 14: Hot-finished structural hollow sections – Dimensions and sectional properties*.

SANS 455, *Covered electrodes for the manual arc welding of carbon and carbon manganese steels*.

SANS 657-1, *Steel tubes for non-pressure purposes – Part 1: Sections for scaffolding, general engineering and structural applications*.

SANS 1200 HC, *Standardized specification for civil engineering construction – Section HC: Corrosion protection of structural steelwork.*

~~SANS 1282 (SABS 1282), *High-strength bolts, nuts and washers for friction-grip joints*.~~

Amdt 1

SANS 1431, *Weldable structural steels*.

SANS 1700 Set (SABS 1700 Set), *Fasteners*

SANS 1921-3, *Construction and management requirements for works contracts – Part 3: Structural steelwork.*

SANS 2001:CS1, *Construction works – Part CS1: Structural steelwork*.

SANS 10094 (SABS 094), *The use of high-strength friction-grip bolts*.

SANS 10100-1 (SABS 0100-1), *The structural use of concrete – Part 1: Design*.

SANS 10100-2 (SABS 0100-2), *The structural use of concrete – Part 2: Materials and execution of work*.

SANS 10160 (all parts), *Basis of structural design and* actions *for buildings and industrial structures*.
Amdt 1

SANS 10162-2 (SABS 0162-2), *The structural use of steel – Part 2: Limit-states design of cold-formed steelwork.*

SANS 14341, *Welding consumables – Wire electrodes and deposits for gas shielded metal arc welding of non alloy and fine grain steels – Classification.* **Amdt 1**

2.2 Publications

South African Steel Construction Handbook: Limit states design, Southern African Institute of steel construction.

Guide to stability design criteria for metal structures, Ed.5, Structural Stability Research Council, John Wiley & Sons.

Journal of the Structural Division, ASCE, Vol. 105, No. ST9, Sept. 1979, Frank and Fisher.

3 Definitions and symbols

3.1 Definitions

For the purposes of this standard, the following definitions apply:

3.1.1
acceptable
acceptable to the Engineer

3.1.2
camber
specified deviation from straightness of a member or structure, to compensate for deflections that will occur in the member or structure when it is loaded (see 6.2.2)

3.1.3
concrete
Portland cement concrete in accordance with SANS 10100-2

3.1.4
engineer
person responsible for the design and satisfactory completion of a structure in accordance with the provisions of this standard

3.1.5
loads

3.1.5.1
design load
the product of the nominal load and the appropriate load factor defined in SANS 10160 (all parts)
Amdt 1

3.1.5.2
fatigue load
design load or action effect pertaining to the fatigue limit state (see clause 26)

3.1.5.3
gravity load
mass of the object being supported, multiplied by the acceleration due to gravity; g

3.1.5.4
nominal load
load as specified in SANS 10160-1 **Amdt 1**

3.1.5.5
serviceability load
design load or action effect pertaining to the serviceability limit state (see SANS 10160-1) **Amdt 1**

3.1.5.6
ultimate load
design load or action effect pertaining to the ultimate limit state (see SANS 10160-1) **Amdt 1**

3.1.6
resistance

3.1.6.1
nominal resistance (*R*)
specified or characteristic resistance or strength of a member, connection or structure, as calculated in accordance with this standard, based on the specified material properties and nominal dimensions

3.1.6.2
ultimate, serviceability or fatigue resistance (ϕR)
product of the nominal resistance and the appropriate resistance factor

3.1.7
resistance factor (ϕ)
factor, given in the appropriate clauses in this standard, applied to a specified material property or the resistance of a member, connection or structure that, for the limit state under consideration, takes into account the variability of material properties, dimensions, workmanship, type of failure, and uncertainty in prediction of member resistance.

NOTE To maintain simplicity of the design formulae in this standard, these uncertainties in the prediction of member resistance have largely been incorporated in the resistance factors

3.1.8
states

3.1.8.1
fatigue limit state
the limiting case of the slow propagation of a crack within a structural element that can result from either live load effects or as the consequence of local distortion within the structure, herein referred to as load-induced or distortion-induced fatigue effects, respectively

3.1.8.2
limit state
condition of a structure at which it ceases to fulfill the function for which it was designed

3.1.8.3
serviceability limit state
state that restricts the intended use and occupancy of the structure and includes excessive deflection, vibration and permanent deformation

3.1.8.4
ultimate limit state
state that concerns safety, including: exceeding the load-carrying capacity, overturning, uplift, sliding, fracture and fatigue failure

3.1.9
structure
any construction (except as qualified in 1.3) consisting primarily of structural steel components, including the detail parts, welds, bolts, fasteners and other items required in the fabrication or erection of the structure

3.1.10
tolerances

3.1.10.1
erection tolerances
tolerances related to the plumbness, alignment and level of the piece as a whole, as specified in SANS 2001:CS1

3.1.10.2
fabrication tolerances
tolerances allowed on the nominal dimensions and geometry, including cutting to length, finishing of ends, cutting of bevel angles and, for fabricated members, out-of-straightness such as bow and camber, as specified in SANS 2001:CS1

3.1.10.3
mill tolerances
variations allowed in the nominal dimensions and geometry, with respect to cross-sectional area, non-parallelism of flanges and out-of-straightness such as bow or camber in the product as manufactured, as summarized in the South African Steel Construction Handbook or as specified in the relevant South African National Standards.

3.1.10.4
sweep
bow
any unintended deviation from straightness in a member or portion thereof, with respect to its minor axis, prior to the application of load

3.2 Symbols

The following symbols are used throughout this standard. Deviations from them and additional nomenclature are noted where they appear.

A_b	=	cross-sectional area of a bolt, based on its nominal diameter
A_c	=	transverse area of concrete between longitudinal shear planes; cross-sectional area of concrete in composite columns
A_{cv}	=	the critical area of two longitudinal shear planes, one on each side of the area A_c, extending from the point of zero moment to the point of maximum moment
A_f	=	flange area
A_g	=	gross area
A_{gv}	=	gross area in shear for block failure
A_m	=	area of fusion face of weld
A_n	=	critical net area; applicable area of parent metal normal to tensile force in partial penetration groove weld
A_{ne}	=	effective net area
A'_{ne}	=	effective net area reduced for shear lag
A_{nt}	=	net area in tension for block failure
A_{nv}	=	net area in shear for block failure
A_p	=	concrete pull-out area
A_r	=	area of reinforcing steel
A_s	=	area of steel section (including cover plates where applicable); area of stiffener or pair of stiffeners
A_{sc}	=	area of steel shear connector
A_{se}	=	effective steel area of a section
A_{st}	=	area of steel in tension
A_v	=	shear area
A_w	=	web area; effective throat area of weld
a	=	distance between the end or the edge and the centre of the fastener hole; depth of concrete compression zone
a'	=	length of cover-plate termination
B	=	bearing force in member or component under serviceability load
B_r	=	factored bearing resistance of member or component
B_u	=	ultimate bearing force in member or component
b	=	width of steel section or plate; design effective width of concrete or cellular slab, width of flange
C	=	compressive force in member or component under serviceability load; axial load
C_e	=	Euler buckling strength = $\pi^2 EI / L^2$
C_{ec}	=	Euler buckling strength of concrete-filled structural hollow section
C_r	=	factored compressive resistance of member of component; factored compressive resistance of steel acting at the centroid of that part of the steel area in compression
C_{rc}	=	factored compressive resistance of composite column
C_r'	=	compressive resistance of concrete acting at the centroid of the concrete area in compression
C_u	=	ultimate compressive force in member or component; ultimate axial load
C_w	=	warping torsional constant
C_y	=	axial compressive force in member at yield stress
c_1	=	coefficient used to determine slip resistance
d	=	diameter; diameter of bolt, stud, rocker or roller; outside diameter of circular hollow section; stiffener factor
E	=	elastic modulus of steel (assumed to be 200 x 10^3 MPa)
E_c	=	elastic modulus of concrete according to SANS 10100-1
e	=	eccentricity; lever arm between compressive resistance C_r and tensile resistance T_r
e'	=	lever arm between compressive resistance C_r of concrete and tensile resistance T_r of steel

F_{st}	=	axial force in stiffener under ultimate load
f	=	unit stress or strength
f_{cr}	=	critical plate-buckling stress in compression, in flexure or in shear
f_{cre}	=	elastic critical plate-buckling stress in shear
f_{cri}	=	inelastic critical plate-buckling stress in shear
f_{cu}	=	specified compressive cube strength of concrete at 28 days in accordance with SANS 10100-1
f_e	=	elastic critical buckling stress in axial compression (see 13.3.1)
f_{fr}	=	fatigue resistance
f_{sr}	=	allowable stress range in fatigue
f_{srt}	=	constant amplitude threshold stress range
f_s	=	ultimate shear stress
f_t	=	tension-field post-buckling stress
f_u	=	specified minimum tensile strength
f_{uw}	=	specified minimum ultimate strength of welding electrode
f_{vu}	=	ultimate shear strength
f_y	=	specified minimum yield stress
f'_y	=	yield stress including effect of cold working
f_{yr}	=	specified minimum yield stress of reinforcing steel
G	=	shear modulus of steel (assumed to be 77 x 10^3 MPa)
g	=	transverse spacing between fastener gauge lines (gauge distance)
h	=	height; depth of steel section; storey height
h_b	=	depth of beam
h_c	=	depth of compression zone
h_d	=	depth of cellular steel deck
h_s	=	height of stud after welding; storey height
h_w	=	clear depth of web between flanges or between web fillets of rolled section
I	=	moment of inertia (subscripts refer to x or y axes)
I_g	=	moment of inertia of cover-plated section
J	=	St. Venant torsion constant of a cross-section
K	=	effective length factor
$K \cdot L$	=	effective length
k	=	distance from outer face of flange to web-toe of fillet of I-section or channel
k_b	=	buckling coefficient
k_s	=	mean slip coefficient
k_v	=	shear buckling coefficient
L	=	gross length; length of member; span of beam; length of longitudinal weld
L_c	=	length of channel shear connector
L_{cr}	=	maximum unbraced length adjacent to a plastic hinge
L_n	=	net length, i.e. gross length less design allowance for holes within the length
M	=	moment; bending moment in member or component under serviceability load
M_{cr}	=	critical elastic moment of laterally unbraced beam
M_p	=	plastic moment = $Z_{pl} \cdot f_y$
M_r	=	factored moment resistance of member or component
M_{rc}	=	factored moment resistance of composite beam; factored moment resistance of column reduced for the presence of axial load
M_u	=	ultimate bending moment in member or component
M_{uc}	=	ultimate bending moment in girder at cut-off point
M_{ug}	=	first-order moment under ultimate gravity loads determined assuming there is no lateral translation of frame (see 8.7)
M_{ut}	=	first-order translational moment under ultimate lateral loads, or moment resulting from lateral translation of asymmetrical frame, or moment resulting in an asymmetrically loaded frame under ultimate gravity loading (see 8.7)
M_{u1}	=	smaller ultimate end moment of beam-column; ultimate bending moment at point of concentrated load
M_{u2}	=	larger ultimate end moment of beam-column

M_y	=	yield moment $= Z_e \cdot f_y$
m	=	number of faying surfaces or shear planes in a bolted joint
N	=	length of bearing of an applied load; number of applications of a moving load
N'	=	number of applications of moving a load at which $f_{sr} = f_{srt}$
N_{fi}	=	number of cycles that would cause failure at stress range level, i (see 26.3.1)
n	=	number; number of bolts; number of shear connectors required between point of maximum positive bending moment and adjacent point of zero moment; number of stress range cycles at a given detail for each application of the moving load
n'	=	number of shear connectors required between any concentrated load and nearest point of zero moment in a region of positive bending moment
P	=	force to be developed in a cover plate; pitch of threads
P_b	=	ultimate force for design of bracing system
P_f	=	axial force
Q_r	=	sum of factored resistances of all shear connectors between points of maximum and zero moment
q_r	=	factored shear resistance of a shear connector
q_{rr}	=	factored shear resistance of shear connector in a ribbed slab
q_{rs}	=	factored shear resistance of shear connector in a solid slab
R	=	nominal resistance of member, connection or structure; beam reaction; transition radius
r	=	radius of gyration
r_y	=	radius of gyration of a member about its weak axis
s	=	centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes; centre-to-centre distance between transverse web stiffeners
T	=	serviceability tensile force in member or component
T_r	=	factored tensile resistance of member or component; in composite construction, factored tensile resistance of the steel acting at the centroid of that part of the steel area in tension
T_u	=	ultimate tensile force in member or component
T_y	=	axial tensile force in member at yield stress
t	=	thickness, thickness of concrete slab
t_f	=	flange thickness; average thickness of tapered or compound flange
t_w	=	web thickness
t'_w	=	sum of thicknesses of column web and doubler plates
U_1	=	factor to account for moment gradient and for second-order effects of axial force acting on the deformed member. An additional subscript, x or y, refers to the direction.
U_2	=	amplification factor to account for second-order effects of gravity loads acting on the laterally displaced storey. An additional subscript, x or y, refers to the direction.
V	=	shear force in member or component under serviceability load
V_h	=	total horizontal shear to be resisted at junction of steel section or joist and slab or steel deck; shear acting at plastic hinge locations when plastic hinging occurs
V_r	=	factored shear resistance of member or component
V_s	=	slip resistance of bolted joint in a friction-grip connection
V_{st}	=	ultimate shear force in column web to be resisted by stiffener
V_u	=	ultimate shear force in member or component
W	=	width-to-thickness ratio, b/t
w	=	plate width
w_d	=	average width of flute of steel deck
w_n	=	net width, i.e. gross width less design allowance for holes within width
x	=	subscript relating to strong axis of section; distance from flange face to centre of plastic hinge
x_u	=	tensile strength of weld metal
$\bar{x}$	=	eccentricity of weld relative to centroid of element
y	=	subscript relating to weak axis of section
Z_e	=	elastic section modulus of steel section
Z_{ef}	=	effective section modulus of steel section
Z_{pl}	=	plastic section modulus of steel section
γ	=	importance factor; fatigue life constant

γ'	=	fatigue life constant at which $f_{sr} = f_{srt}$
Δ_b	=	displacement of bracing system
Δ_o	=	initial misalignment of a member at a brace point
Δ_u	=	relative first-order lateral (translational) displacement of the storey due to ultimate loads (coincident with M_{ut})
κ	=	ratio of smaller to larger ultimate moment at opposite ends of unbraced length, positive for double curvature and negative for single curvature
λ	=	non-dimensional slenderness ratio in column formula
τ	=	composite column coefficient
ρ	=	factor, slenderness ratio
ΣC_u	=	sum of ultimate axial compressive loads of all columns in the storey
ΣV_u	=	sum of ultimate lateral loads above the storey; total first-order storey shear
ϕ	=	resistance factor for structural steel
ϕ_{ar}	=	resistance factor for holding down bolts
ϕ_b	=	resistance factor for bolts
ϕ_{be}	=	resistance factor for beam web bearing, end
ϕ_{bi}	=	resistance factor for beam web bearing, interior
ϕ_{br}	=	resistance factor for bearing of bolts on steel
ϕ_c	=	resistance factor for concrete
ϕ_r	=	resistance factor for reinforcing steel bars
ϕ_{sc}	=	resistance factor for shear connectors
ϕ_w	=	resistance factor for weld metal
ϕ_R	=	ultimate, serviceability or fatigue resistance
ω_1	=	coefficient used to determine equivalent uniform bending effect in beam-columns
ω_2	=	coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

3.3 Units

Equations and expressions used in this standard are compatible with the following SI (metric) units:

– load and force: newtons (N)

– dimensions: millimetres (mm)

– moment: newton-millimetres (N·mm)

– strength and stress: megapascals (MPa)

4 Structural documents

4.1 Structural design documents

4.1.1 The term "documents" includes drawings, standards, computer output, electronic and other data. The documents shall show a complete design of the structure with members suitably designated and located, including such dimensions and detailed description as necessary to permit the preparation of fabrication and erection documents. Floor levels, column centres and offsets shall be dimensioned. Design drawings shall be drawn to a scale adequate to convey the required information.

4.1.2 Structural design documents shall include, but not be limited to, all the relevant information from the following list:

a) the design standards used;

b) the design criteria for snow, wind, seismic or any special loads;

c) the specified live loads for floors, roofs and platforms;

d) the material or product standards (see clause 5);

e) the type or types of construction (see clause 8);

f) the structural system used for seismic design and the seismic zone (see clause 27);

g) the requirements for roof and floor diaphragms;

h) the design criteria for open web steel joists (see clause 16);

i) the design criteria for crane-supporting structures (see annex H);

j) the details of the load-resisting elements necessary to ensure the effectiveness of the load resisting system in the completed structure;

k) the camber of beams, girders and trusses;

l) governing combinations of shears, moments, axial forces, and torsional moments to be resisted by the connections;

m) the size and location of stiffeners, reinforcement and bracing required to stabilize compression elements;

n) the types of bolts, pre-tensioning requirements, and designation of joints as either bearing or friction grip (see clause 22); and

o) any available information to be kept in mind in planning the erection of the structure, to prevent damage or instability.

4.1.3 Revisions to design documents shall be clearly indicated and dated.

4.1.4 Provided all requirements for the structural steel are shown on the structural documents, architectural, electrical and mechanical documents may be used as a supplement to the structural documents to define the detail configurations and construction information.

4.2 Fabrication and erection documents

Fabrication and erection documents shall be in accordance with SANS 2001:CS1.

5 Materials — Standards and identification

5.1 Standards

5.1.1 General

Acceptable material and product standards for use under this standard are listed in 5.1.3 to 5.1.5, inclusive. Materials and products other than those listed may also be used if approved by the Engineer. Approval shall be based on published standards that establish the properties, characteristics and suitability of the material or product to the extent and in the manner as covered in the listed standards.

5.1.2 Design yield stress and tensile strength

The yield stress f_y and the tensile strength f_u used as the basis for design shall be the specified minimum values as given in the relevant material and product standards. The values reported on mill test certificates shall not be used as the basis for design.

5.1.3 Structural steels

Structural steels shall meet the requirements of SANS 1431 (grade 300), EN 10025 (grade 355), or EN 10155 (corrosion-resistant steel), unless otherwise specified by the Engineer.

5.1.4 Special structural steels

Steels complying with recognized standards other than SANS 1431 may be specified by the Engineer or used with his approval, provided that such standards lay down the required chemical composition and mechanical properties of the steel. These steels shall comply with the following requirements:

a) the specified yield stress shall not exceed 700 MPa;

b) the ratio of the minimum tensile strength to the specified yield stress shall be at least 1,2:1; and

c) the elongation (on a gauge length of $\left(5,56\sqrt{A_o}\right)$ mm) in a tensile test shall be at least 15 %, where A_o is the original area of cross-section, in square millimetres.

Where brittle fracture may be a critical design criterion, special measures shall be taken to demonstrate adequate resistance to brittle fracture.
Steel used in welded components shall have a carbon equivalent value not exceeding that given in SANS 1431.

5.1.5 Other materials

Bolts, screws, nuts, washers, welding electrodes and shear studs shall comply with the requirements of SANS 2001:CS1.

5.2 Identification

5.2.1 Methods

The materials and products used shall be identified as specified in SANS 2001:CS1.

5.2.2 Unidentified structural steel

Unidentified structural steel shall not be used unless approved by the Engineer. If the use of unidentified steel is authorized, the minimum yield stress f_y and the minimum tensile strength f_u used for design shall be taken as not more than 200 MPa and 365 MPa, respectively.

5.2.3 Tests to establish identification

Unidentified structural steel may be tested, when permitted by the Engineer, to establish identification. Testing shall be done in accordance with SANS 1431 by a testing agency approved by the Engineer. The test results, taking into account both mechanical properties and chemical composition, shall form the basis for classifying the steel as to the required standard. Once the steel has been classified, the specified minimum values for steel of that standard property class shall be used as the basis for design (see 5.1.2).

6 Design requirements

6.1 General

6.1.1 Limit states

As set out in this standard, structures shall be designed to be safe and serviceable during the useful life of the structure and to be safe from collapse during construction. Limit states define the various types of collapse and unserviceability that are to be avoided; those concerning safety are called the ultimate limit states (which include exceeding of load-carrying capacity, overturning, uplift, sliding, fracture and fatigue failure) and those concerning serviceability are called the serviceability limit states (which include excessive deflection, vibration and permanent deformation). The object of limit-states design calculations is to keep the probability of a limit state being reached below a certain value previously established for the given type of structure. This is achieved in this standard by the use of load factors applied to the specified nominal loads (see clause 7) and resistance factors applied to the specified resistances (see clause 13).
The various limit states are set out in this clause. Some of these relate to the serviceability loads, some to the ultimate loads and others to the fatigue loads. Camber, provisions for expansion and contraction, and corrosion protection are further design requirements related to serviceability and durability. All limit states shall be considered in the design.

6.1.2 Structural integrity

The general arrangement of the structural system and the connection of its members shall be designed to provide resistance to widespread collapse as a consequence of local failure. The requirements of this standard generally provide a satisfactory level of structural integrity for steel structures. Supplementary provisions may be required for structures where accidental loads, for example vehicle impact or chemical explosion, are likely to occur (see 1.3).

6.2 Requirements under serviceability loads

6.2.1 Deflections

6.2.1.1 Members and structures shall be so proportioned that, under serviceability loads, deflections are within acceptable limits for the nature of the materials to be supported and for the intended use or occupancy of the structure.

6.2.1.2 In the absence of a more detailed evaluation, see annex D for recommended values for deflections.

6.2.1.3 Roofs of buildings shall be designed to withstand any loads likely to occur as a result of ponding.

6.2.2 Camber

6.2.2.1 If camber of beams, trusses or girders is required, this shall be specified on the design drawings. Generally, trusses and crane girders of 25 m span or greater should be cambered to compensate for the deflection due to the self-weight load plus half of the imposed load.

6.2.2.2 Any special camber requirements necessary to bring a loaded member into proper relation with the work of other trades shall be specified on the design drawings.

6.2.3 Dynamic effects

6.2.3.1 Suitable provision shall be made in the design for the effect of an imposed load that induces impact or vibration, or both. In severe cases, for example structural supports for heavy machinery that causes substantial impact or vibration when in operation, the possibility of harmonic resonance, fatigue or unacceptable vibration shall be investigated.

6.2.3.2 Special consideration shall be given to floor systems susceptible to vibration, for example large open floor areas free of partitions, to ensure that such vibration is acceptable for the intended use or occupancy. (Guidance regarding floor vibrations is given in annex C.)

6.2.3.3 Unusually flexible structures (generally those whose ratio of height to effective resisting width exceeds 4:1) shall be investigated for lateral vibrations under varying wind load. Lateral accelerations of the structure shall be checked to ensure that such accelerations are acceptable for the intended use or occupancy.

6.3 Requirements under ultimate loads

6.3.1 Strength

Structures shall be designed to resist the effects of ultimate loads as described in 7.2, acting in the most critical combination, including stress reversal, taking into account the resistance factors as specified in the appropriate clauses of this standard.

6.3.2 Overturning and uplift

Structures shall be designed to resist overturning and uplift resulting from the application of the ultimate loads acting in the most critical combination.

6.4 Expansion and contraction

Suitable provision shall be made for expansion and contraction, in a manner commensurate with the service and erection conditions of the structure.

6.5 Corrosion protection

Steelwork shall, where necessary, be adequately protected against corrosion in a manner commensurate with the thickness of material used, the severity of the conditions to which the structure will be exposed and the ease of subsequent inspection and maintenance. Corrosion protection shall be done in accordance with SANS 1200 HC or SANS 2001 CS1.

6.6 Requirements under fatigue loads

Structures shall be designed to resist the effects of fatigue loads in accordance with clause 26.

7 Loads and limit-states criteria

7.1 Loads and actions

Loads and other influences to be considered in the design of a structure shall be in accordance with SANS 10160 (all parts).

Amdt 1

7.2 Limit-states criteria

The criteria of avoiding failure at ultimate or fatigue limit states, or of avoiding unfitness for purpose at serviceability limit states of a structure are, respectively (see also annex B),

a) ultimate resistance ≥ effect of ultimate loads,

b) serviceability requirements ≥ effect of serviceability loads, and

c) fatigue resistance ≥ effect of fatigue loads,

where the ultimate resistance is determined in accordance with other clauses of this standard and the effects of the ultimate, serviceability or fatigue loads are determined in accordance with clause 8, using the ultimate or serviceability loads defined in SANS 10160 (all parts), or the effects of fatigue loads in clause 26. **Amdt 1**

8 Analysis of structure

8.1 General

8.1.1 In proportioning the structure to meet the various design requirements of clause 6, the methods of analysis given in this clause shall be used. The distribution of internal forces and bending moments shall be determined under the serviceability loads to satisfy the requirements for serviceability, under the fatigue loads to satisfy the requirements for resistance to fatigue in clause 26, and under the ultimate loads to satisfy the requirements for strength and resistance to overturning in 6.3.2.

8.1.2 Three basic types of construction and associated design assumptions, designated as "rigidly connected" or "continuous", "simple", and "semi-rigid" are permitted for all or part of a structure under this standard. The distribution of internal forces and bending moments throughout the structure will depend on the type(s) of construction chosen and the forces to be resisted.

8.2 Rigidly connected and continuous construction

In this construction, the beams, girders and trusses shall be rigidly connected to other frame members or be continuous over supports. Connections shall generally be designed to resist the bending moments and internal forces calculated by assuming that the original angles between intersecting members remain unchanged as the structure is loaded.

8.3 Simple construction

In simple construction, it is assumed that the ends of beams, girders and trusses are free to rotate under load in the plane of loading. Resistance to lateral loads, including sway effects, shall be ensured by a suitable system of bracing or shear walls or by the design of part of the structure as rigidly connected or semi-rigid construction.

8.4 Semi-rigid (partially restrained) construction

8.4.1 In this type of construction, the original angles between connected members change under applied bending moments and redistribute the moments between members while maintaining sufficient capacity to resist lateral loads and to provide adequate stability of the framework in accordance with 8.7.

8.4.2 The design and construction of semi-rigid frameworks shall comply with the following:

a) the positive and negative moment/rotation response of the connections up to their maximum capacity shall have been established by test and either published in the technical literature or be available from a reputable testing facility;

b) the design of the structure shall be based on either linear analysis employing the secant stiffness of connections at ultimate load or incremental analyses following the non-linear test response of the connections; and

c) consideration shall be given to the effects of repeated vertical and horizontal loading and load reversals with particular regard to incremental strain in connections and low-cycle fatigue.

8.5 Elastic analysis

Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by an analysis that assumes that individual members behave elastically.

8.6 Plastic analysis

Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by a plastic analysis, provided that

a) the steel used has a specified minimum yield stress of $f_y \leq 0{,}85\ f_u$ and exhibits the load-strain characteristics necessary to achieve moment redistribution,

b) the width-to-thickness ratios meet the requirements for class 1 sections, as given in 11.2,

c) the members are braced laterally in accordance with 13.7,

d) web stiffeners are supplied on a member at a point of load application where a plastic hinge would form,

e) splices in beams or columns are designed to transmit the greater of 1,1 times the maximum calculated moment under ultimate load at the splice location, or 0,25 M_p, whichever is greater,

f) members are not subject to repeated heavy impact or fatigue loading, and

g) the influence of inelastic deformation on the strength of the structure is taken into account. (See also 8.7.)

8.7 Stability effects

The analyses referred to in 8.4, 8.5 and 8.6 shall include the sway effects in each storey that are produced by the vertical loads acting on the structure in its displaced configuration. These second-order effects, due to the relative translational displacement (sway) of the ends of a member, shall preferably be determined from a second-order analysis. Elastic second-order effects may be accounted for by amplifying the translational load effects obtained from a first-order elastic analysis by the factor

$$U_2 = \frac{1}{1 - \left[\dfrac{\Sigma C_u \cdot \Delta_u}{\Sigma V_u \cdot h}\right]}$$

thus, $M_u = M_{ug} + U_2 \cdot M_{ut}$

The translational load effects produced by notional lateral loads, applied at each storey, equal to (0,005 x factored gravity loads contributed by that storey), shall be added to the sway effects for all load combinations.

9 Stability of structures and members

9.1 Stability of structures

The structural system shall be adequate to

a) resist the forces caused by factored loads,

b) transfer the factored loads to the foundations,

c) transfer forces from walls, floors or roofs acting as shear-resisting elements or diaphragms to adjacent lateral-load resisting elements, and

d) resist torsional effects.

(See also 8.7).

9.2 Stability of members

9.2.1 Brace point

A brace point is defined as the point on a member or element at which it is restrained.

9.2.2 Misalignment at brace point

The initial misalignment of the member at a brace point, Δ_o, shall be taken as the maximum tolerance for deviation from a straight line given in SANS 2001:CS1 over the total length between the brace points on either side of the brace being designed.

9.2.3 Displacement at brace point

The displacement of the bracing system at the brace point, Δ_b, is the sum of the brace deformation, the brace connection deformation and the brace support displacement. This displacement is due to the brace force and any other forces acting on the brace and shall be calculated in the direction perpendicular to the braced member at the brace point.

9.2.4 Bracing system

Bracing systems provide lateral support to columns or to the compression flange of beams and girders or to the compression chords of joists or trusses.

Bracing systems include bracing members, their connections and supports, and shall be proportioned to resist the forces that develop at the brace points and limit the lateral displacement of the brace points.

Bracing for beams shall provide lateral restraint to the compression flange, except that for a cantilevered beam at the end of the cantilever and for beams subject to double curvature, the restraint shall be provided at both top and bottom flanges unless otherwise accounted for in the design.

9.2.5 Twisting and lateral displacements

Twisting and lateral displacements shall be prevented at the supports of a member or element unless accounted for in the design.

9.2.6 Simplified analysis

Bracing systems shall be proportioned to have a strength perpendicular to the longitudinal axis of the braced member in the plane of buckling, at least equal to 0,02 times the factored compressive force, at each brace point, in the member or element being braced, unless a detailed analysis is carried out in accordance with 9.2.7 to determine the appropriate strength and stiffness of the bracing system. Any other forces acting on the bracing member shall also be taken into account. The displacement Δ_b shall not exceed Δ_o.

9.2.7 Detailed analysis

9.2.7.1 Second-order method

Forces acting in the member bracing system and its deformations may be determined by means of a second-order elastic analysis of the member and its bracing system. This analysis shall include the most critical initial deformed configuration of the member and shall consider forces due to external loads. In the analysis, hinges may be assumed at brace points in the member or element being braced.

The displacement Δ_b shall not exceed Δ_o unless a greater value can be justified by analysis.

9.2.7.2 Direct method

Unless a second-order analysis is carried out in accordance with 9.2.7.1 or the simplified analysis is carried out in accordance with 9.2.6, bracing systems shall be proportioned at each brace point to have a factored resistance in the direction perpendicular to the longitudinal axis of the braced member in the plane of buckling at least equal to:

$$P_b = \frac{\alpha\left[\Delta_o + \Delta_b\right] C_u}{L}$$

where

P_b is the force used to design the bracing system. When two or more points are braced, the forces P_b alternate in direction;

α is 2; 3; 3,41; 3,63; or 4 for 1, 2, 3, 4, or more equally-spaced braces, respectively, unless a lesser value can be justified by the analysis;

Δ_o is the initial misalignment;

Δ_b is the displacement of the bracing system, assumed to be equal to Δ_o for the initial calculation of P_b;

C_u is the maximum ultimate compression in the segments bound by the brace points on either side of the brace point under consideration;

L is the length between braces.

For flexural members, the force P_b, as calculated above, shall be increased, as appropriate, when loads are applied above the shear centre or for beams in double curvature.

After applying P_b, as calculated above, together with any other forces acting on the bracing member, the calculated displacement of the bracing system, Δ_b, shall not exceed Δ_o unless justified by analysis.

9.2.8 When bracing of the compression flange is effected by a slab or deck, the slab or deck and the means by which the calculated 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. This force taken as at least 0,05 times the maximum force in the flange or chord, unless a lesser amount can be justified by analysis, shall be considered to be uniformly distributed along the length of the compression flange or chord.

9.2.9 Consideration shall be given to the probable accumulation of forces, C_u, when the bracing system restrains more than one member. When members are erected with random out-of-straightness, the initial misalignment may be taken as

$$\left(0{,}2 + 0{,}8/\sqrt{n}\right)\Delta_o$$

where

n is the number of members or elements being braced.

This reduction shall not be applied when initial misalignments of members are dependent on each other and are likely to be in the same direction and of the same magnitude.

9.2.10 Bracing systems for beams, girders and columns designed to resist loads causing torsion shall be proportioned according to the requirements of 14.10. Special consideration shall be given to the connection of asymmetric sections such as channels, angles and Z-sections.

10 Design lengths and slenderness ratios

10.1 Spans of flexural members

10.1.1 Single span flexural members

Beams, girders and trusses may be designed on the basis of simple spans, whose length may be taken as the distance between the centroidal axes of supporting members. Alternatively, the span length of beams and girders may be taken as the actual length of such members measured between centres of end connections. The length of trusses designed as simple spans may be taken as the distance between the extreme working points of the system of triangulation employed. In all cases, the design of columns or other supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder or truss may actually be connected or supported.

10.1.2 Continuous span flexural members

Beams, girders or trusses having full or partial end restraint due to continuity or cantilever action shall be proportioned to carry all moments, shears and other forces at any section assuming the span in general to be the distance between the centres of gravity of the supporting members. Supporting members shall be proportioned to carry all moments, shears and other forces induced by the continuity of the supported beam, girder or truss.

10.2 Effective length factor for flexural members

10.2.1 Simply-supported beams

For simply-supported beams where no lateral restraint of the compression flange is provided but where each end of the beam is restrained against rotation about its longitudinal axis (i.e. against torsion), the effective length factor K to be used in 13.6 shall be as given in table 1.

Table 1 — Effective length factor for simply supported beams

<table>
<tr><td>1</td><td>2</td><td>3</td></tr>
<tr><td rowspan="3">Restraint against lateral bending at support</td><td colspan="2">Effective length factor
K</td></tr>
<tr><td colspan="2">Loading condition</td></tr>
<tr><td>Normal</td><td>Destabilising[a]</td></tr>
<tr><td>Unrestrained (i.e. free to rotate about the vertical axis)</td><td>1,0</td><td>1,2</td></tr>
<tr><td>Partially restrained (i.e. positive connection by flange cleats or end plates)</td><td>0,85</td><td>1,0</td></tr>
<tr><td>Practically fixed (i.e. not free to rotate about the vertical axis)</td><td>0,7</td><td>0,85</td></tr>
<tr><td colspan="3">[a] The destabilising loading condition applies when the load is applied to the compression flange of the beam and both the load and the flange are free to move laterally.</td></tr>
</table>

Restraint against torsion may be provided by

a) web or flange cleats,

b) load-bearing stiffeners acting in conjunction with the bearing of the beam,

c) lateral end frames or other external supports to the ends of the compression flanges, or

d) the flanges being built into walls.

Where the beam ends are not restrained against torsion, the values of the effective length factor in table 1 shall be increased by 20 %.

For beams that are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint as required above, the effective length shall be taken as the distance, centre-to-centre, between the restraint members. The effective lateral restraint shall be as prescribed in 9.2.4.

10.2.2 Cantilever beams

For cantilever beams, the effective length factor K to be used in 13.6 shall be as given in table 2, and L shall be taken as the projecting length.

Table 2 — Effective length factor for cantilever beams

1	2	3	4
Restraint conditions		**Effective length factor** K	
		Loading condition	
At support	**At tip**	**Normal**	**Destabilising**[a]
Built in laterally and torsionally	Free	0,8	1,4
	Lateral restraint only (at compression flange)	0,7	1,4
	Torsional restraint only	0,6	0,6
	Lateral and torsional restraint	0,5	0,5
Continuous, with lateral and torsional restraint	Free	1,0	2,5
	Lateral restraint only (at compression flange)	0,9	2,5
	Torsional restraint only	0,8	1,5
	Lateral and torsional restraint	0,7	1,2
Continuous, with lateral restraint only	Free	3,0	7,5
	Lateral restraint only (at compression flange)	2,7	7,5
	Torsional restraint only	2,4	4,5
	Lateral and torsional restraint	2,1	3,6

[a] The destabilising loading condition applies when the load is applied to the tension flange of the beam and both the load and the flange are free to move laterally.

10.3 Members in compression

10.3.1 General

A member in compression shall be designed on the basis of its effective length, $K \cdot L$ (the product of the effective length factor, K, and the unbraced length, L).

Unless otherwise specified in this standard the unbraced length, L, shall be taken as the length of the compression member between the centres of restraining members. The unbraced length may differ for the different cross-sectional axes of a compression member.

At the bottom storey of a multi-storey structure or for a single-storey structure, L shall be taken as the length from the top of the base plate to the centre of restraining members at the next higher level.

The effective length factor, K, depends on the potential failure modes, whether by bending in-plane or buckling as given in 10.3.2 and 10.3.3.

10.3.2 Failure mode involving bending in-plane

The effective length shall be taken as the actual length ($K = 1,0$) for beam-columns that would fail by in-plane bending provided only that, when applicable, the sway effects, including notional load effects, are included in the analysis of the structure to determine the end moments and forces acting on the beam-columns.

10.3.3 Failure mode involving buckling

The effective length for axially loaded columns that would fail by buckling and for beam-columns that would fail by out-of-plane (lateral-torsional) buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length (see annexes E and F).

10.4 Slenderness ratios

10.4.1 General

The slenderness ratio of a member in compression shall be taken as the ratio of the effective length, $K{\cdot}L$, to the corresponding radius of gyration, r. The slenderness ratio of a member in tension shall be taken as the ratio of the unbraced length, L, to the corresponding radius of gyration.

10.4.2 Maximum slenderness ratio

10.4.2.1 The slenderness ratio of a member in compression shall not exceed 200.

10.4.2.2 The slenderness ratio of a member in tension shall not exceed 300 (see also 15.3.6). This limit may be waived if other means are provided to control flexibility, sag, vibration and slack in a manner commensurate with the service conditions of the structure, or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

11 Width-to-thickness ratios: elements in compression

11.1 Classification of sections

11.1.1 For the purposes of this standard, structural sections shall be designated as class 1, 2, 3, or 4 depending on the maximum width-to-thickness ratios of their elements subjected to compression, and as specified in 11.1.2 and 11.1.3. The classes are defined as follows:

a) class 1 sections will permit attainment of the plastic moment and subsequent redistribution of the bending moment;

b) class 2 sections will permit attainment of the plastic moment but need not allow for subsequent moment redistribution;

c) class 3 sections will permit attainment of the yield moment; and

d) class 4 sections will generally have local buckling of elements in compression as the limit state of structural resistance.

11.1.2 Class 1 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading and, when subject to axial compression, shall be doubly symmetric.

11.1.3 Class 2 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading unless the effects of asymmetry of the section are included in the analysis.

11.2 Maximum width-to-thickness ratios of elements subject to compression

The maximum width-to-thickness ratios, W, of elements subject to axial compression are given in table 3 and those of elements subject to flexural compression are given in table 4 for the specified section classification. Sections with width-to-thickness ratios exceeding the maximum values in table 3 or the class 3 limits in table 4 shall be designated as class 4 sections.

For class 4 sections, see 13.3.3 for factored axial compressive resistance and 13.5 or 13.6 for factored bending resistance.

Table 3 — Maximum width-to-thickness ratios: elements in axial compression

1	2
Description of element	**Maximum width-to-thickness ratio** W
Elements supported along one edge Flanges of I-sections, T-sections, and channels Legs of angles Plate-girder stiffeners	$\frac{b}{t} \leq \frac{200}{\sqrt{f_y}}$
Stems of T-sections	$\frac{b}{t} \leq \frac{340}{\sqrt{f_y}}$
Flanges of rectangular hollow sections Flanges of box sections, Flange cover plates, and diaphragm plates between lines of fasteners or welds Webs supported on both edges	$\frac{b}{t} \leq \frac{670}{\sqrt{f_y}}$
Perforated cover plates	$\frac{b}{t} \leq \frac{840}{\sqrt{f_y}}$
Circular hollow sections	$\frac{d}{t} \leq \frac{23\,000}{f_y}$

Table 4 — Maximum width-to-thickness ratios: elements in flexural compression

1	2	3	4
	Section classification		
Description of element in compression	**Class 1**	**Class 2**	**Class 3**
Flanges of I-sections or T-sections Plates projecting from compression elements Outstanding legs of pairs of angles in continuous contact with an axis of symmetry in the plane of loading	$\frac{b}{t} \le \frac{145}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{170}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{200}{\sqrt{f_y}}$
Stems of T-sections	$\frac{b}{t} \le \frac{145}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{170}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{340}{\sqrt{f_y}}$
Flanges of rectangular hollow sections	$\frac{b}{t} \le \frac{420}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{525}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{670}{\sqrt{f_y}}$
Flanges of box sections Flange cover plates and diaphragm plates between lines of fasteners or welds	$\frac{b}{t} \le \frac{525}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{525}{\sqrt{f_y}}$	$\frac{b}{t} \le \frac{670}{\sqrt{f_y}}$
Webs	$\frac{h_w}{t_w} \le \frac{1100}{\sqrt{f_y}}\left(1 - 0,39\frac{C_u}{\phi \cdot C_y}\right)$	$\frac{h_w}{t_w} \le \frac{1700}{\sqrt{f_y}}\left(1 - 0,61\frac{C_u}{\phi \cdot C_y}\right)$	$\frac{h_w}{t_w} \le \frac{1900}{\sqrt{f_y}}\left(1 - 0,65\frac{C_u}{\phi \cdot C_y}\right)$
Circular hollow sections	$\frac{d}{t} \le \frac{13000}{f_y}$	$\frac{d}{t} \le \frac{18000}{f_y}$	$\frac{d}{t} \le \frac{66000}{f_y}$

11.3 Width and thickness

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

a) for plates, the width, *b*, is the distance from the free edge to the first row of fasteners or line of welds;

b) for legs of angles, flanges of channels and Z-sections and stems of T-sections, the width, *b*, is the full nominal dimension; and

c) for flanges of beams and T-sections, the width, *b*, is one-half of the full nominal dimension.

11.3.2 For elements supported along two edges parallel to the direction of 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 lines of welds;

b) for flanges of rectangular hollow structural sections, the flat width, b, is the nominal outside dimension less four times the wall thickness;

c) for webs of built-up sections, the width, h_w, is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; and for webs of hot-rolled sections, the width, h_w, is the clear distance between flanges.

11.3.3 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.

12 Gross and net areas

12.1 Application

Members in tension shall be proportioned on the basis of the areas associated with the potential failure modes. Members in compression shall be proportioned on the basis of the gross area. For beams and girders, see clause 14.

12.2 Gross area

Gross areas shall be calculated by summing the products of the thickness and the gross width of each element (flange, web, leg or plate), as measured normal to the axis of the member.

12.3 Net area and effective net area

12.3.1 The effective net area, $A_{ne,}$ shall be determined by summing the critical net areas, A_n, of each segment along a potential path of minimum resistance calculated as follows:

a) for a segment normal to the force (i.e., in direct tension)

$$A_{ne} = w_n \cdot t$$

b) for a segment inclined to the force

$$A_{ne} = w_n \cdot t + \frac{s^2 \cdot t}{4g}$$

12.3.2 In calculating w_n, the width of bolt holes shall be taken as 2 mm larger than the specified hole diameter. Where it is known that drilled or punched-and-drilled holes will be used, this allowance may be waived.

12.3.3 Effective net area–shear lag

12.3.3.1 When fasteners transmit load to each of the cross-sectional elements of a member in tension in proportion to their respective areas, the effective net area is equal to the net area i.e.,

$$A'_{ne} = A_{ne}$$

12.3.3.2 When bolts transmit load to some but not all of the cross-sectional elements and only when the critical net area includes the net area of unconnected elements, the effective net area shall be taken as follows:

a) for all sections with I or H shapes with flange widths not less than two-thirds the depth, and for structural tees cut from these sections, when only the flanges are connected with three or more transverse lines of fasteners,

$$A'_{ne} = 0{,}90\ A_{ne}$$

b) for angles connected by only one leg with

i) four or more transverse lines of fasteners,

$$A'_{ne} = 0{,}80\ A_{ne}$$

ii) fewer than four transverse lines of fasteners,

$$A'_{ne} = 0{,}60\ A_{ne}$$

c) for all other structural sections connected with

i) three or more transverse lines of fasteners,

$$A'_{ne} = 0{,}85\ A_{ne}$$

ii) with two transverse lines of fasteners,

$$A'_{ne} = 0{,}75\ A_{ne}$$

12.3.3.3 When a tension load is transmitted by welds, the effective net area shall be computed as

$$A'_{ne} = \left(A_{ne1} + A_{ne2} + A_{ne3}\right) \leq A_{ne}$$

where A_{ne1}, A_{ne2} and A_{ne3} are the net areas of the connected plate elements subject to one of the following methods of load transfer:

a) for elements connected by transverse welds:

$$A_{ne1} = w \cdot t$$

b) for elements connected by longitudinal welds along two parallel edges:

i) when $L \geq 2w$,

$$A_{ne2} = w \cdot t$$

ii) when $2w > L \geq w$,

$A_{ne2} = 0{,}50\ w \cdot t + 0{,}25 L \cdot t$

iii) when $w > L$,

$A_{ne2} = 0{,}75\ L \cdot t$

where

L is the average length of welds on the two edges;

w is the plate width (distance between longitudinal welds);

c) for elements connected by a single longitudinal weld, A_{ne3}

i) when $L \geq w$,

$$A_{ne3} = \left(1 - \frac{\bar{x}}{L}\right) w \cdot t$$

ii) when $w > L$,

$$A_{ne3} = 0{,}50 \cdot L \cdot t$$

where

$\bar{x}$ is the eccentricity of the weld with respect to the centroid of the connected element;

L is the length of weld in the direction of the loading.

The outstanding leg of an angle is considered connected by the (single) line of weld along the heel.

12.3.3.4 Larger values of the effective net area may be used if justified by test or rational analysis.

12.3.4 For angles, the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

12.3.5 In calculating the net area of a member across plug or slot welds, the weld metal shall not be taken as adding to the net area.

12.4 Pin-connected members in tension

12.4.1 In pin-connected members in tension, the net area, A_n, across the pin hole, normal to the axis of the member, shall be at least 1,33 times the minimum required cross-sectional area of the body of the member. The net area of any section on either side of the axis of the member, measured at an angle of 45° or less to the axis of the member, shall be not less than 0,9 times the minimum required cross-sectional area of the body of the member.

12.4.2 The distance from the edge of the pin hole to the edge of the member, measured transverse to the axis of the member, shall not exceed four times the thickness of the material at the pin hole.

12.4.3 The diameter of a pin hole shall not be more than 1 mm larger than the diameter of the pin.

13 Member and connection resistance

13.1 Resistance factors

Unless otherwise specified, resistance factors, ϕ, applied to resistances given in this standard shall be taken as follows:

a) structural steel, ϕ = 0,90;

b) reinforcing steel bars, ϕ_r = 0,85;

c) bolts, ϕ_b = 0,80;

d) shear connectors, ϕ_{sc} = 0,80;

e) beam web bearing, interior, ϕ_{bi} = 0,80;

f) beam web bearing, end, ϕ_{be} = 0,75;

g) bearing of bolts on steel, ϕ_{br} = 0,67;

h) weld metal, ϕ_w = 0,67;

i) holding down bolts, ϕ_{ar} = 0,67; and

j) concrete, ϕ_c = 0,60.

The factored resistances so determined, in order to meet the strength requirements of this standard, shall be greater than or equal to the effect of the ultimate loads as defined in 7.2.

13.2 Axial tension member and connection resistance

The factored tensile resistance, T_r, developed by a member subjected to an axial tensile force shall be taken as

a) the least of

i) $T_r = \phi \cdot A_g \cdot f_y$

ii) $T_r = 0{,}85\ \phi \cdot A_{ne} \cdot f_u$

iii) $T_r = 0{,}85\ \phi \cdot A'_{ne} \cdot f_u$

and

b) for pin connections,

$$T_r = 0{,}75\ \phi \cdot A_n \cdot f_u$$

13.3 Axial compression

13.3.1 Flexural buckling

The factored axial compressive resistance, C_r, of doubly symmetric sections conforming to the requirements of clause 11 for class 1, 2, or 3 sections shall be taken as

$$C_r = \phi \cdot A \cdot f_y (1 + \lambda^{2n})^{-1/n}$$

where

n = 1,34 for hot-rolled, fabricated structural sections, and hollow structural sections manufactured according to SANS 657-1 (cold-formed non-stress-relieved); or

2,24 for doubly symmetric welded three-plate members with flange edges oxy-flame-cut and hollow structural sections manufactured according to ISO 657-14 (hot-formed or cold-formed stress-relieved).

$$\lambda = \frac{K \cdot L}{r}\sqrt{\frac{f_y}{\pi^2 \cdot E}} = \sqrt{\frac{f_y}{f_e}}$$

Doubly symmetric sections which may be governed by torsional flexural buckling shall also meet the requirements of 13.3.2.

13.3.2 Torsional or torsional-flexural buckling

The factored compressive resistance, C_r, of asymmetric, singly symmetric, and cruciform or other bisymmetric sections not covered under 13.3.1 shall be computed using the expressions given in 13.3.1 with a value of n = 1,34 and the value of f_e taken as

a) for doubly symmetric (e.g., cruciform) and axisymmetric (e.g., Z-sections), the least of f_{ex}, f_{ey}, and f_{ez};

b) for singly symmetric sections, with the y axis taken as the axis of symmetry, the lesser of f_{ex} and f_{eyz},

where

$$f_{eyz} = \frac{f_{ey} + f_{ez}}{2\,\Omega}\left(1 - \sqrt{1 - \frac{4 f_{ey} \cdot f_{ez} \cdot \Omega}{\left(f_{ey} + f_{ez}\right)^2}}\right)$$

where

$$f_{ey} = \frac{\pi^2 \cdot E}{\left(\frac{K_y \cdot L_y}{r_y}\right)^2} \quad \text{and}$$

$$f_{ez} = \left(\frac{\pi^2 \cdot E \cdot C_w}{K_z^2 \cdot L_z^2} + G \cdot J\right)\frac{1}{A \cdot \bar{r}_o^2}$$

$$\Omega = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^2} \right)$$

where

x_0, y_0 are the principal coordinates of the shear centre with respect to the centroid of the cross-section.

$$\bar{r}_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2$$

c) For asymmetric sections, f_e is the smallest root of

$$\left(f_e - f_{ex}\right)\left(f_e - f_{ey}\right)\left(f_e - f_{ez}\right) - f_e^2 \left(f_e - f_{ey}\right)\left(\frac{x_o}{\bar{r}_o}\right)^2 - f_e^2 \left(f_e - f_{ex}\right)\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0$$

where

$$f_{ex} = \frac{\pi^2 \cdot E}{\left(\frac{K_x \cdot L_x}{r_x}\right)^2}$$

13.3.3 Class 4 members in compression

The factored compressive resistance, C_r, for sections that are designated as class 4 sections according to clause 11 shall be determined by calculating the slenderness ratios of members using their gross section properties and an effective area which is calculated as follows:

a) when $W \leq W_{lim}$

$A_{ef} = A$

where

A_{ef} is the effective area (see 13.3.3b));

$$W = \frac{b}{t};$$

$$W_{lim} = 0{,}644\sqrt{\frac{k \cdot E}{f}}$$

where

f is the calculated compressive stress in the element ($\leq f_y$), using ultimate loads and gross section properties;

k = 4,0 for elements supported along both longitudinal edges; or

0,43 for elements supported along one longitudinal edge only.

When

$W > W_{lim}$

A_{ef} shall be determined using an effective area calculated with reduced element widths meeting either the maximum width-to-thickness ratios of class 3 sections or, where applicable, an element width equal to b to be taken as

$$b = 0{,}95t\sqrt{\frac{k \cdot E}{f}}\left(1 - \frac{0{,}208}{W}\sqrt{\frac{k \cdot E}{f}}\right)$$

13.4 Shear

13.4.1 Webs of flexural members with two flanges

13.4.1.1 Elastic analysis

Except as noted in 13.4.1.2, the factored shear resistance, V_r, developed by the web of a flexural member, shall be taken as

$$V_r = \phi \cdot A_v \cdot f_s$$

where

A_v is the shear area (commonly $t_w \cdot h_w$ for plate girders and $t_w \cdot h$ for rolled steel sections);

f_s is as follows:

a) $f_s = 0{,}66\, f_y$ when $\frac{h_w}{t_w} \leq 440\sqrt{\frac{k_v}{f_y}}$,

where

k_v is the shear buckling coefficient defined as:

i) $k_v = 4 + \frac{5{,}34}{(s/h_w)^2}$ if $s/h_w < 1$

ii) $k_v = 5{,}34 + \frac{4}{(s/h_w)^2}$ if $s/h_w \geq 1$

b) $f_s = f_{cri}$ when $440\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w} \leq 500\sqrt{\frac{k_v}{f_y}}$,

where

$f_{cri} = 290\frac{\sqrt{f_y \cdot k_v}}{(h_w/t_w)}$;

k_v is as defined in (a);

c) $f_s = f_{cri} + f_t$ when $500\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w} \leq 620\sqrt{\frac{k_v}{f_y}}$,

where

f_t is the tension-field post-buckling stress defined as:

$f_t = k_a(0{,}50\ f_y - 0{,}866\ f_{cri})$

where

k_a is the aspect coefficient defined as:

$$= \frac{1}{\sqrt{1 + (s/h_w)^2}}$$

d) $f_s = f_{cre} + f_t$ when $620\sqrt{\frac{k_v}{f_y}} < \frac{h_w}{t_w}$,

where

f_t is tension-field post-buckling stress defined as:

$f_t = k_a(0{,}50 f_y - 0{,}866\ f_{cre})$

where

$$f_{cre} = \frac{180\ 000\ k_v}{(h_w/t_w)^2}$$

k_v is as defined in (a).

13.4.1.2 Plastic analysis

In structures designed on the basis of a plastic analysis as defined in 8.6, the factored shear resistance, V_r, developed by the web of a flexural member subjected to shear shall be taken as

$$V_r = 0{,}55\ \phi \cdot t_w \cdot h \cdot f_y$$

13.4.2 Webs of flexural members not having two flanges

The factored shear resistance for cross-sections not having two flanges (e.g. solid rectangles, rounds or tees) shall be determined by rational analysis. The shear stress at ultimate load at any location in the cross-section shall not exceed $0{,}66\phi f_y$ and shall be reduced where shear buckling is a consideration.

13.4.3 Pins

The total factored shear resistance of the nominal area of pins shall be taken as

$$V_r = 0{,}66\ \phi A \cdot f_y$$

13.5 Bending — Laterally supported members

The factored moment resistance, M_r, developed by a member subjected to uni-axial bending moments about a principal axis and where continuous lateral support is provided to the compressive flange shall be taken as

a) for class 1 and class 2 sections

$$M_r = \phi \cdot Z_{pl} \cdot f_y = \phi \cdot M_p$$

b) for class 3 sections

$$M_r = \phi \cdot Z_e \cdot f_y = \phi \cdot M_y$$

c) for class 4 sections

i) when both the web and the compressive flange exceed the limits for class 3 sections, the value of M_r shall be determined in accordance with SANS 10162-2; the calculated value, f'_y, applicable to cold-formed members, shall be determined by using only the values for f_y and f_u that are specified in the relevant structural steel material standard,

ii) for beams or girders whose flanges meet the requirements of class 3 and whose webs exceed the limits for class 3, see clause 14,

iii) for beams or girders whose webs meet the requirements of class 3 and whose flanges exceed the limits for class 3, the moment resistance shall be calculated as

$$M_r = \phi \cdot Z_{ef} \cdot f_y$$

where

Z_{ef} is the effective section modulus determined using an effective flange width of $670\, t_f \big/ \sqrt{f_y}$ for flanges supported along two edges parallel to the direction of stress and an effective width of $200\, t_f \big/ \sqrt{f_y}$ for flanges supported along one edge parallel to the direction of stress. For flanges supported along one edge, in no case shall b/t exceed 60.

Alternatively, the moment resistance may be calculated using an effective yield stress determined from the width-to-thickness ratio meeting the class 3 limit.

13.6 Bending — Laterally unsupported members

Where continuous lateral support is not provided to the compression flange of a member subjected to uni-axial strong axis bending, the factored moment resistance, M_r, may be taken as follows:

a) for doubly symmetric classes 1 and 2 sections, except closed square and circular sections

i) when $M_{cr} > 0{,}67\, M_p$

$$M_r = 1{,}15\, \phi . M_p \left(1 - \frac{0{,}28\ M_p}{M_{cr}} \right) \text{ but not greater than } \phi \cdot M_p;$$

ii) when $M_{cr} \leq 0{,}67\, M_p$

$$M_r = \phi \cdot M_{cr}$$

where the critical elastic moment of the unbraced member is given by

$$M_{cr} = \frac{\omega_2 \cdot \pi}{K \cdot L} \sqrt{E \cdot I_y \cdot G \cdot J + \left(\frac{\pi \cdot E}{K \cdot L}\right)^2 I_y \cdot C_w}$$

where

$K \cdot L$ is the effective length of the unbraced portion of beam, in millimetres;

ω_2 = $1,75 + 1,05\kappa + 0,3\kappa^2 \leq 2,5$ for unbraced lengths subject to end moments; or

= 1,0 when the bending moment at any point within the unbraced length is larger than the larger end moment or when there is no effective lateral support for the compression flange at one of the ends of the unsupported length

where

κ = is the ratio of the smaller factored moment to the larger factored moment at opposite ends of the unbraced length, positive for double curvature and negative for single curvature;

C_w = 0,0 for hollow structural sections;

b) for doubly symmetric class 3 and class 4 sections, except closed square and circular sections, and for channels

i) when $M_{cr} > 0,67\ M_y$

$$M_r = 1,15\phi \cdot M_y \left(1 - \frac{0,28 M_y}{M_{cr}}\right)$$

but not greater than ϕM_y for class 3 sections and the value given in 13.5 (c) (iii) for class 4 sections

ii) when $M_{cr} \leq 0,67\ M_y$

$M_r = \phi M_{cr}$

with M_{cr} and ω_2 as defined in 13.6(a);

c) for closed square and circular sections, M_r shall be determined in accordance with 13.5;

d) for monosymmetric sections, a rational method of analysis such as that given in *the Structural Stability Research Council's Guide to Stability Design Criteria for Metal Structures* should be used; and

e) for biaxial bending, the member shall meet the following criterion:

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1,0$$

13.7 Lateral bracing of members in structures analysed plastically

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 displacement at all hinge locations. However, bracing is not required at the location of the last hinge to form in the failure mechanism assumed as the basis for proportioning the structure. The laterally unsupported distance, L_{cr}, from braced hinge locations to the nearest adjacent point on the frame similarly braced shall not exceed the calculated value for L_{cr} , obtained with:

$$L_{cr} = \left[\frac{25\,000 + 15\,000\,\kappa}{f_y} \right] r_y$$

Except for the aforementioned regions, the maximum unsupported length of members in structures analysed plastically need not be less than that permitted for the same members in structures analysed elastically.

13.8 Axial compression and bending

13.8.1 General

In this clause a distinction is made between braced and unbraced frames. A frame with direct acting bracing is classified as braced when its sway stiffness is at least five times that of the frame without direct acting bracing.

13.8.2 Member strength and stability — class 1 and class 2 I-shaped sections

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_u}{C_r} + \frac{0{,}85 U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{\beta \cdot U_{1y} \cdot M_{uy}}{M_{ry}} \leq 1{,}0$$

where

C_u and M_u are the maximum load effects, including stability effects as defined in 8.7;

$\beta = 0{,}6 + 0{,}4\lambda_y \leq 0{,}85$

where

λ_y is the non-dimensional slenderness parameter about the y-y axis.

The capacity of the member shall be examined for

a) cross-sectional strength (members in braced frames only), with $\beta = 0{,}6$, in which case

C_r is as defined in 13.3 with the value of $\lambda = 0$,
M_r is as defined in 13.5 (for the appropriate class of section), and
U_{1x} and U_{1y} are as defined in 13.8.4 but not less than 1,0, and

b) overall member strength, in which case,

C_r is as defined in 13.3 with the value of $K = 1$, except that for uniaxial strong-axis bending,
C_r $= C_{rx}$ (see also 10.3.2),
M_r is as defined in 13.5 (for the appropriate class of section),
U_{1x} and U_{1y} are as defined in 13.8.4 for members in braced frames, and
U_{1x} and U_{1y} are taken as 1,0 for members in unbraced frames, and

c) lateral torsional buckling strength, when applicable, in which case

C_r is as defined in 13.3, and is based on weak-axis or torsional-flexural buckling (see also 10.3.3),
M_{rx} is as defined in 13.6 (for the appropriate class of section),
M_{ry} is as defined in 13.5 (for the appropriate class of section),
U_{1x} and U_{1y} are taken as 1,0 for members in unbraced frames,
U_{1x} is as defined in 13.8.4 but not less than 1,0, for members in braced frames, and
U_{1y} is as defined in 13.8.4 for members in braced frames.

In addition, the member shall meet the following criterion:

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1{,}0$$

where

M_{rx} and M_{ry} are as defined in 13.5 or 13.6, as appropriate.

13.8.3 Member strength and stability — All classes of sections except class 1 and class 2 I-shaped sections

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}} \leq 1{,}0$$

where all terms are as defined in 13.8.2.

The capacity of the member shall be examined for the following cases in a parallel manner to that in 13.8.2:

a) cross-sectional strength (members in braced frames and tapered members only);

b) overall member strength; and

c) lateral-torsional buckling strength.

13.8.4 Value of U_1

In lieu of a more detailed analysis, the value of U_1 for the axis under consideration, accounting for the second-order effects due to the deformation of a member between its ends, shall be taken as

$$U_1 = \frac{\omega_1}{1 - C_u / C_e}$$

where

ω_1 for the axis under consideration is defined in 13.8.4;

$C_e = \frac{\pi^2 \cdot E \cdot I}{L^2}$ for the axis under consideration.

13.8.5 Values of ω_1

Unless otherwise determined by analysis, the following values shall be used for ω_1:

a) for members not subjected to transverse loads between supports

$$\omega_1 = 0{,}6 - 0{,}4\kappa \geq 0{,}4$$

b) for members subjected to distributed loads or a series of point loads between supports

$$\omega_1 = 1{,}0$$

c) for members subjected to a concentrated load or moment between supports

$$\omega_1 = 0{,}85.$$

For the purpose of design, members subjected to concentrated load or moment between supports (e.g., crane columns) may be considered to be divided into two segments at the point of load (or moment) application. Each segment shall then be treated as a member that depends on its own flexural stiffness to prevent side-sway in the plane of bending considered, and ω_1 shall be taken as 0,85. In calculating the slenderness ratio for use in 13.8, the total length of the member shall be used.

13.9 Axial tension and bending

Members required to resist both bending moments and an axial tensile force shall be proportioned so that

a) $$\frac{T_u}{T_r} + \frac{M_u}{M_r} \leq 1{,}0$$

where

M_r is $\phi \cdot M_p$ for class 1 and class 2 sections;

M_r is $\phi \cdot M_y$ for class 3 and class 4 sections; and

b) $$\frac{M_u}{M_r} - \frac{T_u \cdot Z_{pl}}{M_r \cdot A} \leq 1{,}0 \text{ for class 1 and class 2 sections;}$$

$$\frac{M_u}{M_r} - \frac{T_u \cdot Z_e}{M_r \cdot A} \leq 1{,}0 \text{ for class 3 and class 4 sections}$$

where

M_r is defined in 13.5 or 13.6.

13.10 Load bearing

The factored bearing resistance, B_r, developed by a member or portion of a member subjected to bearing shall be taken as follows:

a) on the contact area of machined, accurately sawn, or fitted parts

$$B_r = 1{,}50 \cdot \phi \cdot A\ f_y$$

b) on expansion rollers or rockers

$$B_r = 0{,}000\ 26\ \phi \left[\frac{R_1}{1 - R_1/R_2} \right] L \cdot f_y^2$$

where

B_r is given in newtons;

L is the length of the roller or rocker (mm);

R_1 is the radius of the roller or rocker (mm);

R_2 is the radius of the groove of the supporting plate (mm);

f_y is the specified minimum yield point of the weaker part in contact (MPa);

c) in bolted connections with n bolts, the lesser of

$B_r = 3\phi_{br} \cdot t \cdot d \cdot n \cdot f_u$, and

$B_r = \phi_{br} \cdot a \cdot t \cdot n \cdot f_u$

where

ϕ_{br} is taken as 0,67;

f_u is the tensile strength of the connected material (MPa);

a is the end distance from centre of hole (mm).

NOTE See also 13.2 and 13.11 for resistances for bolted parts and 22.3 for limiting end distance/bolt diameter ratios.

13.11 Tension and shear block failure

The factored resistance of a connected part whose failure mode involves both tensile fracture and either shear yielding or shear fracture shall be taken as:

a) for gusset plates, for angle cleats and single plate connections, as well as the ends of tension members, the lesser of

i) $T_r + V_r = \phi \cdot A_{nt} \cdot f_u + 0{,}60\ \phi \cdot A_{gv} \cdot f_y$, or

ii) $T_r + V_r = \phi \cdot A_{nt} \cdot f_u + 0{,}60\ \phi \cdot A_{nv} \cdot f_u$, and

b) for notched beams the lesser of

i) $T_r + V_r = 0{,}50\phi \cdot A_{nt} \cdot f_u + 0{,}60\ \phi \cdot A_{gv} \cdot f_y$, or

ii) $T_r + V_r = 0{,}50\phi \cdot A_{nt} \cdot f_u + 0{,}60\ \phi \cdot A_{nv} \cdot f_u$

where

A_{nt} is the net area in tension for block failure;

A_{gv} is the gross area in shear for block failure;

A_{nv} is the net area in shear for block failure.

13.12 Bolts

13.12.1 Bolts in bearing-type connections

13.12.1.1 General

For bolts subject to shear or tension, ϕ_b shall be taken as 0,80.

13.12.1.2 Bolts in shear

The factored resistance developed by a bolted joint subjected to shear shall be taken as the lesser of

a) the factored bearing resistance, B_r, given in 13.10(c); or

b) the factored shear resistance of the bolts, which shall be taken as

$$V_r = 0,60\phi_b \cdot n \cdot m \cdot A_b \cdot f_u$$

When the bolt threads are intercepted by any shear plane, the factored shear resistance of any joint shall be taken as $0,70V_r$.

For lap splices with $L \geq 15d$, where d is the bolt diameter and L is the joint length between centres of end fasteners, the shear resistance of the bolts shall be taken as $(1,075 - 0,005\ L/d)V_r$, but not less than $0,75\ V_r$.

NOTE The specified minimum tensile strength, f_u, for bolts is given in the SANS 1700 series, and may be taken as 420 MPa for class 4.8, 830 MPa for class 8.8, and 1 040 MPa for class 10.9 bolts or screws.

13.12.1.3 Bolts in tension

The factored tensile resistance developed by a bolt in a joint subjected to tensile force, shall be taken as

$$T_r = 0,75\ \phi_b \cdot A_b \cdot f_u$$

The calculated tensile force, T_u, is independent of the pretension and shall be taken as the sum of the external load plus any tension caused by prying action.

A high-strength bolt subjected to tensile cyclic loading shall be pretensioned as for friction grip connections (see clause 22). Connected parts shall be arranged so that prying forces are minimized, and in no case shall the prying force exceed 30 % of the externally applied load. The permissible range of stress under specified loads, based on the shank area of the bolt, shall not exceed 214 MPa for class 8.8S bolts or 262 MPa for class 10.9S bolts.

In lieu of calculating the actual fatigue stress range, which requires the effect of bolt pretension to be taken into account, the stress range may be simply taken as the calculated stress based on the shank area of the bolt under specified loads, including any prying force, and independent of the pretension force.

13.12.1.4 Bolts in combined shear and tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that

$$\left(\frac{V_u}{V_r}\right) + \left(\frac{T_u}{T_r}\right) \leq 1{,}4$$

where

V_r is given in 13.12.1.2 and T_r is given in 13.12.1.3.

13.12.2 Bolts in friction grip connections

13.12.2.1 General

The requirement for a friction grip connection where round holes are used is that under the forces and moments produced by serviceability loads, slip of the assembly shall not occur. Where slotted holes are used, the consequences of slip shall be considered when determining whether slip of the assembly shall not occur under the forces and moments produced by ultimate or serviceability loads. In addition, the effects of ultimate loads shall not exceed the resistances of the connection as given in 13.12.1.

13.12.2.2 Shear connections

The slip resistance, V_s, of a bolted joint, subjected to shear, V_u, shall be taken as

$$V_s = 0{,}53 c_1 \cdot k_s \cdot m \cdot n \cdot A_b \cdot f_u$$

where

k_s is the mean slip coefficient as determined by tests carried out in accordance with SANS 10094 (see table 5);

c_1 is a coefficient that relates the specified initial tension and mean slip to a 5 % probability of slip (see table 5).

When long slotted holes are used in friction grip connections the value of V_s shall be taken as 0,75 of the aforementioned value.

Table 5 — Values of k_s and c_1 for friction grip connections

1	2	3	4	5	6
Coatings of bolted parts		**Mean slip coefficient** k_s	c_1		
			Turn-of-the-nut method of installation		**Torque control method of installation (not recommended)**
Class	**Description**		**Class 8.8S bolts**	**Class 10.9S bolts**	**Class 8.8S or 10.9S bolts**
A	Clean mill scale (not polished by wire brushing), or blast-cleaned with class A coatings	0,33	0,82	0,78	0,72
B	Blast-cleaned or blast-cleaned with class B coatings	0,50	0,90	0,85	0,79
C	Hot-dip galvanized wire brushed or blast cleaned	0,40	0,90	0,85	0,78
NOTE Class A and class B coatings are defined as those coatings that provide a mean slip coefficient, k_s, of not less than 0,33 and 0,50, respectively.					

13.12.2.3 Connections in combined shear and tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that the following relationship is satisfied for the specified loads:

$$\frac{V}{V_s} + 1{,}9\,\frac{T}{n \cdot A_b \cdot f_u} \leq 1{,}0$$

where

V_s is the slip resistance as defined in 13.12.2.2

13.13 Welds

13.13.1 General

The resistance factor, ϕ_w, for welded connections shall be taken as 0,67. Matching electrode classifications for grades 300 WA and S355JR steels are given in table 6.

13.13.2 Shear

13.13.2.1 Complete and partial joint penetration groove welds, plug and slot welds

Table 6 — Approved parent metal — Weld metal combinations for SANS 1431 steels

1	2	3	4	5	6	7
Parent metal				**Weld metal**		
Grade	**Minimum yield strength** MPa	**Minimum ultimate tensile strength** MPa	**Process**	**Electrode**	**Minimum yield strength** MPa	**Minimum ultimate tensile strength** MPa
300 WA	300	450	SMAW (Shielded metal arc welding)	AWS A5.1 E70XX	365	480
				AWS A5.5 E70XX-X	390	480
			SAW (Submerged arc welding)	AWS A5.17 F7XX - EXXX	400	480 / 650
			GMAW (Gas metal arc welding)	AWS A5.18 ER70S -X	400	480
			FCAW (Flux cored arc welding)	AWS A5.20 E7XT – X	400	480
				AWS 5.29 E7XTX-X	400	490
S355JR	355	480	SMAW (Shielded metal arc welding)	AWS A5.1 E7015, E7016 D7018, E7028	400	480
				AWS A5.5 E7015-X, E7016-X E7018-X	390	480
			SAW (Submerged arc welding)	AWS A5.17 F7XX – EXXX	400	480
				AWS 5.3 E7XX – EXX - XX	400	480
			GMAW (Gas metal arc welding)	AWS A5.18 ER70S - X	400	480
			FCAW (Flux cored arc welding)	E7XT - X (except –2, -3, -10, -13, -14, GS)	400	480
				AWS 5.29 E7X T X-X	400	490

The factored shear resistance shall be taken as the lesser of

a) for the base metal

$V_r = 0{,}67\phi_w \cdot A_m \cdot f_u$

where

A_m is the shear area of the effective fusion face (mm^2);

f_u is the tensile strength of the parent metal (MPa);

b) for the weld metal

$V_r = 0{,}67 \cdot \phi_w \cdot A_w \cdot x_u$

where

A_w is the area of the effective weld throat, plug or slot (mm^2);

x_u is the tensile strength of the weld metal (MPa).

13.13.2.2 Fillet welds

The factored resistance for shear and tension-shear or compression-induced shear shall be taken as the lesser of

a) for the base metal

$V_r = 0{,}67\phi_w \cdot A_m \cdot f_u$; or

b) for the weld metal

$V_r = 0{,}67\phi_w \cdot A_w \cdot x_u\ (1{,}00 + 0{,}50 \sin^{1,5}\theta)$

where

θ is the angle of the axis of the weld with the line of action of the force (0° for a longitudinal weld and 90° for a transverse weld), and the other terms are defined in 13.13.2.1. Conservatively, $(1{,}00 + 0{,}50 \sin^{1,5}\theta)$ can be taken as 1,0.

13.13.3 Tension normal to axis of weld

13.13.3.1 Complete joint penetration groove (CJPG) weld made with matching electrodes

The factored tensile resistance shall be taken as that of the base metal.

13.13.3.2 Partial joint penetration groove (PJPG) weld made with matching electrodes

The factored tensile resistance shall be taken as:

$$T_r = \phi_w \cdot A_n \cdot f_u \leq \phi \cdot A_g \cdot f_y$$

where

A_n is the nominal area of fusion face normal to the tensile force (mm^2).

When overall ductile behaviour is desired (member yielding before weld fracture), $A_n \cdot f_u > A_g \cdot f_y$.

13.13.3.3 Partial joint penetration groove weld combined with a fillet weld, made with matching electrodes

The factored tensile resistance shall be taken as

$$T_r = \phi_w \sqrt{(A_n \cdot f_u)^2 + (A_w \cdot x_u)^2} \le \phi \cdot A_g \cdot f_y$$

where

A_g is the gross area of the components of the tension member connected by the welds (mm^2).

13.13.4 Compression normal to axis of weld

13.13.4.1 Complete and partial joint penetration groove welds, made with matching electrodes

The compressive resistance shall be taken as that of the effective area of base metal in the joint. For partial joint penetration groove welds, the effective area in compression is the nominal area of the fusion face normal to the compression plus the area of the base metal fitted in the contact bearing.

13.13.4.2 Cross-sectional properties of continuous longitudinal welds

All continuous longitudinal welds, made with matching electrodes, may be considered as contributing to the cross-sectional properties, A, Z_e, Z_{pl} and I.

14 Beams and girders

14.1 Proportioning

Beams and girders consisting of rolled sections (with or without cover plates), hollow structural sections or fabricated sections shall be proportioned on the basis of the properties of the gross section or the modified gross section. No deduction need be made for fastener holes in webs or flanges unless the reduction of flange area by such holes exceeds 15 % of the gross flange area, in which case the excess shall be deducted. The effect of openings other than holes for fasteners shall be considered in accordance with 14.3.3.

14.2 Flanges

14.2.1 Flanges of welded girders preferably shall consist of a single plate or a series of plates joined end-to-end by complete penetration groove welds.

14.2.2 Flanges of bolted girders shall be proportioned so that the total cross-sectional area of cover plates does not exceed 70 % of the total flange area.

14.2.3 Fasteners or welds connecting flanges to webs shall be proportioned to resist horizontal shear forces due to bending combined with any loads that are transmitted from the flange to the web other than by direct bearing. Spacing of fasteners or intermittent welds in general shall be in proportion to the intensity of the shear force and shall not exceed the maximum for compression or tension members as applicable, in accordance with clause 19.

14.2.4 Partial-length flange cover plates shall be extended beyond the theoretical cut-off point, and the extended portion shall be connected with sufficient fasteners or welds to transmit a force, *P*, in the cover plate at the theoretical cut-off point not less than

$$P = \frac{A \cdot M_{uc} \cdot y}{I_g}$$

where

A is the area of cover plate (mm^2);

M_{uc} is the moment due to factored loads at the theoretical cut-off point (MPa);

y is the distance from the centroid of the cover plate to the neutral axis of the cover-plated section (mm^2);

I_g is the moment of inertia of the cover-plated section.

Additionally, for welded cover plates, the welds connecting the cover-plate termination to the beam or girder shall be designed to develop the force, *P*, within a length, *a*', measured from the actual end of the cover plate, determined as follows:

a) *a*' is the width of the cover plate when there is a continuous weld equal to or larger than three-fourths of the cover-plate thickness across the end of the plate and along both edges in the length *a*';

b) *a*' is 1,5 times the width of the cover plate when there is a continuous weld smaller than three-fourths of the cover-plate thickness across the end of the plate and along both edges in the length *a*'; and

c) *a*' is 2 times the width of the cover plate when there is no weld across the end of the plate but there are continuous welds along both edges in the length *a*'.

14.3 Webs

14.3.1 Maximum slenderness

The slenderness ratio (h_w/t_w) of a web shall not exceed 83 000/f_y

where

f_y is the specified minimum yield stress of the compression flange steel.

This limit may be waived if analysis indicates that buckling of the compression flange into the web will not occur at factored load levels.

14.3.2 Web crippling and yielding

The factored bearing resistance of the web, B_r, shall be taken as follows:

a) for interior loads (concentrated load applied at a distance from the member end greater than the member depth), the smaller of

i) $B_r = \phi_{bi} \cdot t_w \cdot f_y$ (N + 10 t_f), or

ii) $B_r = 1{,}45\ \phi_{bi} \cdot t_w^2 \sqrt{f_y \cdot E}$; and

b) for end reactions, the smaller of

i) $B_r = \phi_{be} \cdot t_w \cdot f_y \left(N + 4t_f\right)$, or

ii) $B_r = 0{,}60\ \phi_{be} \cdot t_w^2 \sqrt{f_y \cdot E}$

where

N is the length of bearing (mm²);

ϕ_{bi} is 0;80;

ϕ_{be} is 0;75.

Wherever the bearing resistance of the web is exceeded, see 14.4.

14.3.3 Openings

14.3.3.1 Except as provided in 14.1, the effect of all openings in beams and girders shall be considered in the design. At all points where the factored shear or moments at the net section would exceed the capacity of the member, adequate reinforcement shall be added to the member at that point to provide the required strength and stability.

14.3.3.2 Unreinforced circular openings may be located in the web of unstiffened prismatic class 1 and class 2 beams or girders without considering net section properties provided that

a) the load is uniformly distributed;

b) the section has an axis of symmetry in the plane of bending;

c) the openings are located within the middle third of the depth and the middle half of the span of the member;

d) the spacing between the centres of any two adjacent openings, measured parallel to the longitudinal axis of the member, is a minimum of 2,5 times the diameter of the larger opening; and

e) the factored maximum shear at the support does not exceed 50 % of the factored shear resistance of the section.

14.3.3.3 If the forces at openings are determined by an elastic analysis, the procedure adopted shall be in accordance with published, recognized principles.

14.3.3.4 The strength and stability of the member in the vicinity of openings may be determined on the basis of assumed locations of plastic hinges, such that the resulting force distributions satisfy the requirements of equilibrium, provided that the analysis is carried out in accordance with 8.6(a), (b), and (f). However, for I-shaped members the width-to-thickness ratio of the flanges may meet the requirements of class 2 sections, provided that the webs meet the width-to-thickness limit of class 1 sections.

14.3.4 Effect of thin webs on moment resistance

When the web slenderness ratio, h_w/t_w, exceeds $1\,900/\sqrt{M_u/\phi \cdot Z_e}$, the flange shall meet the width-to-thickness ratios of class 3 sections in accordance with clause 11 and the factored moment resistance of the beam or girder, M'_r, shall be determined by

$$M'_r = M_r\left[1 - 0{,}0005\frac{A_w}{A_f}\left(\frac{h_w}{t_w} - \frac{1900}{\sqrt{M_u/\phi \cdot Z_e}}\right)\right]$$

where

M_r is the factored moment resistance as determined by 13.5 or 13.6 but not to exceed ϕM_y.

When an axial compressive force acts on the girder in addition to the moment, the constant 1 900 in the expression for M'_r shall be reduced by the factor $(1 – 0{,}65\ C_u/\phi C_y)$. See also 11.2.

14.4 Bearing stiffeners

14.4.1 Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the bearing resistance of the web is exceeded (see 14.3.2). Bearing stiffeners shall also be required at the ends of single-web girders having web depth-to-thickness ratios greater than $1\,100/\sqrt{f_y}$ where the tension field is not adequately anchored. Box girders may employ diaphragms designed to act as bearing stiffeners.

14.4.2 Bearing stiffeners shall bear against the flange or flanges through which they receive their loads and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with 13.3, assuming the column section to consist of the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at interior stiffeners, or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, $K \cdot L$, shall not be taken as less than three-fourths of the length of the stiffeners in calculating the ratio $K \cdot L/r$. Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web so as to develop the full force required to be carried by the stiffener into the web or vice versa.

14.5 Intermediate transverse stiffeners

14.5.1 Intermediate transverse stiffeners, when used, shall be spaced to suit the shear resistance determined in accordance with 13.4, except that at girder end panels or at panels adjacent to large openings, the tension-field component shall be taken as zero unless means are provided to anchor the tension field.

14.5.2 The maximum distance between stiffeners, when stiffeners are required, shall not exceed the values shown in table 7. Closer spacing may be required in accordance with 14.5.1.

Table 7 — Maximum intermediate transverse stiffener spacing

1	2
Web depth-to-thickness ratio h_w/t_w	**Maximum distance between stiffeners, *s*, in terms of clear web depth, h_w** mm
Up to 150	$3h_w$
Over 150	$\dfrac{67\,500\,h_w}{(h_w/t_w)^2}$

14.5.3 Intermediate transverse stiffeners may be furnished singly or in pairs. Width-to-thickness ratios shall conform to clause 11. The moment of inertia of the stiffener, or pair of stiffeners if so furnished, shall be not less than $(h_w/50)^4$ taken about an axis in the plane of the web. The gross area, A_s, of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be

$$A_s \geq \frac{s \cdot t_w}{2}\left\{1 - \frac{s/h_w}{\sqrt{1+(s/h_w)^2}}\right\} C \cdot Y \cdot D$$

where

s is the centre-to-centre distance of adjacent stiffeners (i.e., panel length) (mm);

$$C = \left[1 - \frac{310\ 000 k_v}{f_y (h_w/t_w)^2}\right] \text{ but not less than 0,10}$$

Y is the ratio of the specified minimum yield point of the web steel to the specified minimum yield point of the stiffener steel;

D is the stiffener factor which is

1,0 for stiffeners furnished in pairs; or

1,8 for single angle stiffeners; or

2,4 for single plate stiffeners;

k_v is the shear buckling coefficient (see 13.4.1.1);

f_y is the specified minimum yield stress of the web steel.

When the ultimate shear force, V_u, in an adjacent panel is less than that permitted by 13.4.1, the gross area requirement may be reduced by multiplying it by the ratio V_u/V_r.

14.5.4 Intermediate transverse stiffeners shall be connected to the web for a shear transfer per pair of stiffeners (or per single stiffener when so furnished), of not less than $1 \times 10^{-4} h_w \cdot f_y^{1,5}$ newtons per millimetre of web depth, h_w, except that when the largest calculated shear, V_u, in the adjacent panels is less than V_r, this shear transfer may be reduced in the same proportion. However, the total shear transfer shall in no case be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300 mm from centre to centre. If intermittent fillet welds are used, the clear distance between welds shall not exceed 16 times the web thickness or 4 times the weld length.

14.5.5 When intermediate stiffeners are used on only one side of the web, the stiffeners shall be attached to the compression flange. Intermediate stiffeners used in pairs shall have at least a snug fit against the compression flange. When stiffeners are cut short of the tension flange, the distance cut short shall be equal to or greater than four times, but not greater than six times, the girder web thickness. Stiffeners preferably shall be clipped to clear girder flange-to-web welds.

14.6 Combined shear and moment

Transversely stiffened girders depending on tension-field action to carry shear shall be proportioned such that

$$0{,}727\,\frac{M_u}{M_r} + 0{,}455\,\frac{V_u}{V_r} \leq 1{,}0$$

and

$$\frac{M_u}{M_r} \leq 1{,}0 \text{ and } \frac{V_u}{V_r} \leq 1{,}0 .$$

where

V_r is established in accordance with 13.4;

M_r is established in accordance with 13.5 or 13.6, as applicable.

14.7 Rotational restraint at points of support

Beams and girders shall be restrained against rotation about their longitudinal axes at points of support, or comply with 10.2.

14.8 Notches

14.8.1 The effect of notches on the lateral torsional buckling resistance of a beam or girder shall be taken into account.

14.8.2 The effect of notches in reducing the net area of the web available to resist transverse shear and the effective net area of potential paths of minimum resistance shall be taken into account (see 13.11).

14.9 Lateral forces

The flanges of beams and girders supporting cranes or other moving loads shall be proportioned to resist any lateral forces produced by such loads.

14.10 Torsion

14.10.1 Beams and girders subjected to torsion shall have sufficient strength and rigidity to resist the torsional moment and forces in addition to other moments or forces. The connections and bracing of such members shall be adequate to transfer the reactions to the supports.

14.10.2 The factored resistance of I-section members subject to combined flexure and torsion may be determined from moment-torque interaction diagrams that take into account the normal stress distribution due to flexure and warping torsion and the St. Venant torsion. Assumed normal stress distributions shall be consistent with the class of section.

14.10.3 Members subject to torsional deformations required to maintain compatibility of the structure need not be designed to resist the associated torsional moments provided that the structure satisfies the requirements of equilibrium.

14.10.4 For all members subject to loads causing torsion, the torsional deformations under specified loads shall be limited in accordance with the requirements of 6.2.1. For members subject to torsion or to combined flexure and torsion, the maximum combined normal stress, as determined by an elastic analysis, arising from warping torsion and bending due to the specified loads shall not exceed f_y.

15 Trusses

15.1 General

A truss is a triangulated framework primarily loaded in flexure. Trusses designed to act compositely with the deck slab shall also meet the requirements of clause 17.

15.2 Analysis

15.2.1 Simplified method

The simplified method assumes that all members are pin-connected and loads are only applied at the panel points, except that bending effects due to transverse loads applied between panel points shall be assessed by taking into account any continuity of the members. This method may be used when:

a) the out-of-plane resistance of all compression members is larger than the in-plane resistance; and

b) the compression members are at least class 3.

15.2.2 Detailed method

The detailed method accounts for the actual loading and joint fixity. The detailed method shall be used for trusses

a) with panels adjacent to abrupt changes in the slope of a chord;

b) with Vierendeel panels;

c) with panels at abrupt changes in transverse shear; and

d) for fatigue design.

15.3 General requirements

15.3.1 Effective lengths of compression members

The effective length for buckling in the plane of the truss shall be taken as the distance between the lines of intersection of the working points of the web members and the chord. The effective length for buckling perpendicular to the plane of the truss shall be equal to the distance between the points of lateral support. For built-up members, see clause 19.

15.3.2 Joint eccentricities

Bending moments due to joint eccentricities shall be taken into account in the design of members and end connections.

15.3.3 Stability

Trusses shall be braced to ensure their lateral stability. Brace members that support compression chords at discrete points shall meet the requirements of 9.2. Ends of compression chords that are not attached to a supporting member shall be braced laterally, unless it can be demonstrated that the support is not necessary.

15.3.4 Web members

The factored resistances of the first compression web member and its connections shall be determined with their respective resistance factors, ϕ, multiplied by 0,85.

15.3.5 Compression chord supports

Truss web members that provide support to a compression chord in the plane of the truss shall be designed for an additional force equal to 0,02 of the chord force, unless such forces have been determined by rigorous analysis.

15.3.6 Maximum slenderness ratio of tension chords

The maximum slenderness ratio shall be limited to 240, except if other means are provided to control flexibility, sag, vibration and slack in a manner commensurate with the service conditions of the structure.

15.3.7 Deflection and camber

Except for the deflection due to flexural deformation of Vierendeel panels, deflections may be determined from the axial deformations of the truss members. For camber, see 6.2.2.

16 Open-web steel joists

Open-web steel joists and associated decking shall meet the requirements of CSA S16.

17 Composite beams, trusses and joists

17.1 General

17.1.1 Application

The provisions of clause 17 apply to composite beams consisting of steel sections, trusses or open-web joists interconnected with either a reinforced concrete slab or a steel deck with a concrete cover slab. Trusses and open-web joists designed to act compositely with the deck slab shall also meet the requirements of clause 15 and clause 16, respectively.

17.1.2 Resistance factors

In this clause, the resistance factors shall be taken as ϕ = 0,90, ϕ_c = 0,60 and ϕ_r = 0,85, except when stated otherwise.

17.1.3 Resistance prior to composite action

The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with clause 13.

17.2 Definitions

The following definitions apply to clause 17:

17.2.1
cover slab
concrete above the flutes of the steel deck

17.2.2
effective slab thickness (*t*)
overall slab thickness, minus the height of the flute corrugation

17.2.3
flute
portion of the steel deck that forms a valley

NOTE Flutes filled with concrete form a ribbed slab and the thickness of the cover slab should be at least 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests.

17.2.4
rib
portion of the concrete slab that is formed by the steel deck flute

17.2.5
slab
reinforced cast-in-situ concrete slab at least 65 mm in effective thickness

NOTE The area equal to the effective width times the effective slab thickness should be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness

17.2.6
steel deck
load-carrying steel deck, consisting of either
a) a single fluted element (non-cellular deck), or
b) a two-element section consisting of a fluted element in conjunction with a flat sheet (cellular deck)

NOTE The maximum depth of the deck shall be 80 mm and the average width of the minimum flute shall be 50 mm. A steel deck may be of a type intended to act compositely with the cover slab in supporting the applied load.

17.2.7
steel joist
open-web steel joist suitable for composite design. (See clause 16.)

17.2.8
steel section
structural steel section with a solid web or webs suitable for composite design

NOTE Web openings are permissible only on condition that their effects are fully investigated and accounted for in the design.

17.2.9
steel truss
steel truss suitable for composite design. (See clause 15.)

17.3 Effects

17.3.1 Deflections

Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to the effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analysis, the following calculations may be used to assess the effects of partial shear connection and interfacial slip, creep and shrinkage:

a) for increased flexibility resulting from partial shear connection and interfacial slip, calculate the deflections using an effective moment of inertia, I_e, given by

$$I_e = I_s + 0,85\, p^{0,25} \left(I_t - I_s\right)$$

where

I_s is the moment of inertia of the steel beam or of a steel joist or truss adjusted to include the effect of shear deformations, which may be taken into account by decreasing the moment of inertia based on the cross-sectional areas of the top and bottom chords by 15 %, or by a more detailed analysis;

I_t is the transformed moment of inertia of the composite beam;

p is a fraction of the full shear connection (use p = 1,0 for full shear connection).

b) for creep, increase elastic deflections caused by self-weight loads and long-term imposed loads (as calculated in (a) above) by 15 %; and

c) for shrinkage of concrete, calculate deflection using a selected shrinkage strain, strain compatibility between the steel and concrete, and a time-dependent modulus of elasticity of the concrete in tension, E_{ct}, (see annex G) as it dries, shrinks, and creeps from

$$\Delta_s = \frac{\varepsilon_f \cdot A_c \cdot L^2 \cdot y}{8 n_t \cdot I_t}$$

where

ε_f is the free shrinkage strain of the concrete;

A_c is the effective area of the concrete slab;

E_{ct} is the effective modulus of concrete in tension;

L is the span of the beam;

n_t is the modular ratio, E/E_{ct};

y is the distance from the centroid of effective area of the concrete slab to the elastic neutral axis;

I_t is the transformed moment of inertia of the composite beam but based on the modular ratio n_t.

17.3.2 Vertical shear

The web area of steel sections or the web system of steel trusses and joists shall be proportioned to carry the total vertical shear, V_u.

17.3.3 End reaction

End reactions of steel sections, trusses and joists shall be proportioned to transmit the total end reaction of the composite beam.

17.4 Design effective width of concrete

17.4.1 Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, b, equal to the lesser of

a) 0,25 times the composite beam span; or

b) the average distance from the centre of the steel section, truss or joist to the centres of adjacent parallel supports.

17.4.2 Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, b, not greater than the width of the top flange of the steel section or top chord of the steel joist or truss plus the lesser of

a) 0,1 times the composite beam span; or

b) 0,5 times the clear distance between the steel section, truss or joist and the adjacent parallel support.

17.5 Slab reinforcement

17.5.1 General

Slabs shall be adequately reinforced to support all loads and to control both cracking transverse to the composite beam span and longitudinal cracking over the steel section or joist. Reinforcement shall not be less than that required by the specified fire-resistance design of the assembly or that required to limit shrinkage cracking.

17.5.2 Parallel reinforcement

Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete that is in compression. The reinforcement of slabs that are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention. In no case shall such reinforcement at the ends of beams supporting ribbed slabs perpendicular to the beam be less than two Y16 bars or equivalent to two Y16 bars.

17.5.3 Transverse reinforcement, solid slabs

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section or joist is unlikely, additional transverse reinforcement or other effective means shall be provided. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0,002 times the concrete area being reinforced and shall be uniformly distributed.

17.5.4 Transverse reinforcement, ribbed slabs

17.5.4.1 Where the ribs are parallel to the beam span, the area of transverse reinforcement shall be not less than 0,002 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.5.4.2 Where the ribs are perpendicular to the beam span, the area of transverse reinforcement shall be not less than 0,001 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.6 Interconnection

17.6.1 Except as permitted by 17.6.2 and 17.6.4, interconnection between steel sections, trusses, or joists and slabs or steel decks with cover slabs shall be attained by the use of shear connectors in accordance with 17.7.

17.6.2 Uncoated steel sections, trusses or joists that support slabs and are totally encased in concrete do not require interconnection by means of shear connectors provided

a) that a minimum of 50 mm of concrete covers all portions of the steel section, truss or joist except as noted in (c) below;

b) the cover in (a) above is reinforced to prevent spalling; and

c) the top of the steel section, truss or joist is at least 4 mm below the top and 5 mm above the bottom of the slab.

17.6.3 Studs may be welded through a maximum of two steel sheets in contact, each not more than 1,71 mm in overall thickness including coatings (1,5 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise, holes for placing studs shall be made through the sheets as necessary. Welded studs shall comply with AWS D1.1.

17.6.4 Other methods of interconnection that have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss or joist and the slab or steel deck with cover slab.

In such cases the design of the composite member shall conform to the design of a similar member employing shear connectors, insofar as practicable.

17.6.5 The diameter of a welded stud shall not exceed 2,5 times the thickness of the part to which it is welded, unless test data that satisfy the designer are provided to establish the capacity of the stud as a shear connector.

17.7 Shear connectors

17.7.1 General

The resistance factor, ϕ_{sc} , to be used with the shear resistances given in this clause shall be taken as 0,80. The factored shear resistance, q_r, of other shear connectors shall be established by tests acceptable to the designer.

17.7.2 End-welded studs

End-welded studs shall be headed or hooked with $h_s/d \geq 4$. The projection of a stud in a ribbed slab, based on its length prior to welding, shall be at least two stud diameters above the top surface of the steel deck.

17.7.2.1 In solid slabs factored shear resistance of shear connector, q_{rs}, is:

$$q_{rs} = 0{,}45\ \phi_{sc} \cdot A_{sc} \sqrt{f_{cu} \cdot E_c} \quad \leq \phi_{sc} \cdot A_{sc} \cdot f_u \quad \text{(N)}$$

where

f_u for commonly available studs is 415 MPa;

E_c is the short-term modulus of elasticity determined in accordance with SANS 10100-1.

17.7.2.2 In ribbed slabs with ribs parallel to the beam

a) when $w_d/h_d \geq 1{,}50$

$$q_{rr} = q_{rs}$$

b) when $w_d/h_d < 1{,}50$

$$q_{rr} = \phi_{sc}\left(0{,}77\frac{w_d}{h_d}d \cdot h_s (f_{cu})^{0{,}8} + 10{,}5 s \cdot d\,(f_{cu})^{0{,}2}\right) \leq q_{rs}$$

where

s is the longitudinal stud spacing.

17.7.2.3 In ribbed slabs with ribs perpendicular to the beam

a) when h_d = 75 mm

$$q_{rr} = 0{,}31\ \phi_{sc} \cdot \rho \cdot A_p \cdot \sqrt{f_{cu}} \leq q_{rs}$$

b) when h_d = 38 mm

$$q_{rr} = 0{,}55\ \phi_{sc} \cdot \rho \cdot A_p \cdot \sqrt{f_{cu}} \leq q_{rs}$$

where

A_p is the concrete pull-out area taking the deck profile and stud burn-off into account. For a single stud, the apex of the pyramidal pull-out area, with four sides sloping at 45°, is taken as the centre of the top surface of the head of the stud. For a pair of studs, the pull-out area has a ridge extending from stud to stud;

ρ = 1,0 for normal-density concrete (2 150 to 2 500 kg/m^3); or

0,85 for semi-low-density concrete (1 850 to 2 150 kg/m^3).

17.7.2.4 The longitudinal spacing of stud connectors in both solid slabs and in ribbed slabs when ribs of formed steel deck are parallel to the beam shall be not less than six stud diameters. The maximum longitudinal spacing of studs shall not exceed 1 000 mm. See also 17.8. The transverse spacing of stud connectors shall not be less than four stud diameters.

17.7.3 Channel connectors

In solid slabs of normal-density concrete with $f_{cu} \geq 25$ MPa and a density of at least 2 300 kg/m^3,

$$q_{rs} = 32{,}6\ \phi_{sc} \cdot L_c \left(t_f + 0{,}5\, t_w\right) \sqrt{f_{cu}}$$

where

t_f is the flange thickness of the channel;

t_w is the web thickness of the channel.

17.8 Ties

Mechanical ties shall be provided between the steel section, truss or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1 000 mm, and the average spacing in a span shall not exceed 600 mm or be greater than that required to achieve any specified fire-resistance rating of the composite assembly.

17.9 Design of composite beams with shear connectors

17.9.1 The composite beam shall consist of the steel section, truss or joist, shear connectors, ties and slab or steel deck with cover slab.

The flat width of the top chord or that of a component member of the top chord shall not be less than 1,4d + 20 mm

where

d is the diameter of the stud connector.

17.9.2 The properties of the composite section shall be based on the maximum effective area (equal to effective width times effective thickness) neglecting any concrete area in tension. If a steel truss or joist is used, the area of its top chord shall be neglected in determining the properties of the composite section and only 17.9.3(a) is applicable.

17.9.3 The factored moment resistance, M_{rc}, of the composite section with the slab or cover slab in compression shall be calculated as follows, where ϕ = 0,90 and the resistance factor for concrete, ϕ_c = 0,60:

a) **case 1** – Full shear connection and plastic neutral axis in the slab;

that is, $Q_r \geq \phi \cdot A_s \cdot f_y$ and $\phi \cdot A_s \cdot f_y \leq 0{,}68\phi_c \cdot b \cdot t \cdot f_{cu}$ where Q_r equals the sum of the factored resistances of all shear connectors between points of maximum and zero moment

$$M_{rc} = T_r e' = \phi \cdot A_s \cdot f_y \cdot e'$$

where

e' is the lever arm, based on the depth of the concrete compression zone being equal to a, where

$$a = \frac{\phi \cdot A_s \cdot f_y}{0{,}68\phi_c \cdot b \cdot f_{cu}}$$

b) **case 2** – Full shear connection and plastic neutral axis in the steel section;

that is $Q_r \geq 0{,}68\phi_c \cdot b \cdot t \cdot f_{cu}$ and $0{,}68\phi_c \cdot b \cdot t\, f_{cu} < \phi \cdot A_s \cdot f_y$

$$M_{rc} = C_r \cdot e + C_r' \cdot e'$$

$$C_r' = 0{,}68\phi_c \cdot b \cdot t \cdot f_{cu}$$

$$C_r = \frac{\phi \cdot A_s \cdot f_y - C_r'}{2}$$

c) **case 3** — Partial shear connection; that is, $Q_r < 0{,}68\phi_c \cdot b \cdot t \cdot f_{cu}$ and $< \phi \cdot A_s \cdot f_y$

$$M_{rc} = C_r \cdot e + C_r' \cdot e'$$

$$C_r' = Q_r$$

$$C_r = \frac{\phi \cdot A_s \cdot f_y - C_r'}{2}$$

where e' is the lever arm, based on the depth of the concrete compression zone being equal to a,

and

$$a = \frac{C_r'}{0{,}68\phi_c \cdot b \cdot f_{cu}}$$

17.9.4 No composite action shall be assumed in calculating flexural strength when Q_r is less than 0,4 times the lesser of 0,68 $\phi_c \cdot b \cdot t \cdot f_{cu}$ and $\phi \cdot A_s \cdot f_y$. No composite action shall be assumed in calculating deflections when Q_r is less than 0,25 times the lesser of $0{,}68\phi_c \cdot b \cdot t \cdot f_{cu}$ and $\phi \cdot A_s \cdot f_y$.

17.9.5 For full shear connection, the total horizontal shear, V_h, at the junction of the steel section, truss or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment shall be the lesser of

$$V_h = \phi A_s \cdot f_y$$

and

$$V_h = 0{,}68\ \phi_c \cdot b \cdot t \cdot f_{cu}$$

for cases 1 and 2 as defined in 17.9.3(a) and (b), respectively, and where $Q_r \geq V_h$.

17.9.6 For partial shear connection the total horizontal shear, V_h, shall be

$$V_h = Q_r$$

where Q_r is as defined in 17.9.3(c).

17.9.7 Composite beams employing steel sections and concrete slabs may be designed as continuous members. The factored moment resistance of the composite section, with the concrete slab in the tension area of the composite section, shall be the factored moment resistance of the steel section alone, except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in calculating the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and each adjacent point of zero moment shall be taken as $\phi_r \cdot A_r \cdot f_{yr}$, where ϕ_r = 0,85.

17.9.8 The number of shear connectors to be located on each side of the point of maximum bending moment (positive or negative, as applicable), distributed between that point and the adjacent point of zero moment, shall be not less than

$$n = \frac{V_h}{q_r}$$

Shear connectors may be spaced uniformly, except that in a region of positive bending the number of shear connectors, n', required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than

$$n' = \left(\frac{M_{u1} - M_r}{M_u - M_r} \right)$$

where

M_{u1} is the positive bending moment under ultimate load at the concentrated load point;

M_r is the factored moment resistance of the steel section alone;

M_u is equal to the maximum positive bending moment under ultimate load.

17.9.9 In the end panels of composite joists and trusses, the top chord shall be designed to resist all factored forces, ignoring any composite action unless adequate shear connectors are placed over the seat or along a top chord extension to carry horizontal shear. Studs shall not be placed closer than their height to the end of the concrete slab.

17.9.10 Longitudinal shear

The shear that is to be developed on the longitudinal shear surfaces, A_{cv}, of composite beams with solid slabs or with cover slabs and steel deck parallel to the beam, shall be taken as

$$V_u = \Sigma\, q_r - 0{,}68\phi_c \cdot A_c \cdot f_{cu} - \phi_r \cdot A_r \cdot f_{yr}$$

where

A_r/ is the area of longitudinal reinforcement within the concrete area, A_c.

For normal-weight concrete, the factored shear resistance along any potential longitudinal shear surfaces in the concrete slab shall be taken as

$$V_r = (0{,}80\phi_r \cdot A_{rt} \cdot f_{yr} + 2{,}76\phi_c \cdot A_{cv}) \leq 0{,}40\phi_c \cdot A_{cv} \cdot f_{cu}$$

where

A_{rt} is the area of transverse reinforcement crossing the shear planes, A_{cv}.

17.10 Design of composite beams without shear connectors

17.10.1 Uncoated steel sections or joists supporting concrete slabs and encased in concrete in accordance with 17.6.2 may be proportioned on the basis that the composite section supports the total load.

17.10.2 The properties of the composite section for determining the load carrying capacity shall be calculated by ultimate strength methods, neglecting any area of concrete in tension.

17.10.3 As an alternative method of design, encased simple-span steel sections or joists may be proportioned on the basis that the steel section, truss or joist alone supports 0,90 times the total load.

17.11 Unpropped beams

For composite beams that are unpropped during construction, the stresses in the tension flange of the steel section, truss or joist due to the loads applied before the concrete strength reaches 0,75 f_{cu}, plus the stresses at the same location due to the remaining specified loads considered to act on the composite section, shall not exceed f_y.

17.12 Serviceability considerations at extreme levels of stress

For composite beams in which the total serviceability load exceeds the minimum load, P_d, required to cause an elastic stress of either 0,9 f_y in the steel or 0,4 f_{cu} in the concrete, or a moment of 0,75 M_u, the total serviceability deflection, Δ_{serv}, shall be calculated as follows:

$$\Delta_{serv} = \Delta_s \left(1 - EI_s / EI_c\right) + \Delta_{dc} \left(P_{serv} / P_d\right)^2$$

where

Δ_s is the elastic deflection due to serviceability load applied to plain steel section;

EI_s is the flexural rigidity of the plain steel section;

EI_c is the flexural rigidity of the composite section;

Δ_{dc} is the elastic deflection due to the total load, P_d, applied to the composite section and including shrinkage;

P_{serv} is the total serviceability load in the range $P_d < P_{serv} < 1{,}225 P_d$.

17.13 Beams propped during construction

The steel section, truss or joist alone shall be proportioned to support all factored loads applied prior to the concrete achieving 75 % of the specified strength, without exceeding its calculated capacity under the conditions of lateral support or vertical propping, or both, that will apply during construction.

18 Composite columns

18.1 General

18.1.1 Resistance factors

In this clause, the resistance factors shall be taken as ϕ = 0,90, ϕ_c = 0,60 and ϕ_r = 0,85, except when stated otherwise.

18.1.2 Resistance prior to composite action

The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with clause 13.

18.1.3 Surface finish

The surface of a steel member in contact with concrete shall be uncoated.

18.2 Concrete filled hollow structural sections

18.2.1 Scope

18.2.1.1 The provisions of this clause apply to composite members consisting of rectangular or circular steel hollow structural sections completely filled with concrete.

The design procedure applies within the following limitations:

a) the width-to-thickness ratio of the walls of rectangular hollow structural sections shall not exceed $1\,350 / \sqrt{f_y}$;

b) the outside diameter-to-thickness ratios of circular hollow structural sections shall not exceed $28\,000/f_y$; and

c) the concrete strength shall be between 25 MPa and 100 MPa for axially loaded columns and between 25 MPa and 50 MPa for columns subjected to axial compression and bending.

18.2.1.2 Axial load on concrete

The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on concrete. Similarly, a base plate or other means shall be provided for load transfer at the bottom of a column. At intermediate floor levels, direct bearing on the concrete is not necessary.

18.2.1.3 Composite action in bending

Full composite bending resistance as specified in 18.2.3 can be developed at the ends of concrete-filled hollow structural members in bending or combined axial load and bending, such as at column bases, only if the connection is able to transfer the forces from both the steel and concrete elements to the adjacent structural elements.

18.2.2 Compressive resistance

In the case of a composite concrete-filled hollow structural section the factored compressive resistance shall be taken as

$$C_{rc} = (\tau \cdot \phi \cdot A_s \cdot f_y + 0,68\tau' \cdot \phi_c \cdot A_c \cdot f_{cu})(1+\lambda^{2n})^{-1/n}$$

where

n = 1,80;

τ is a composite column coefficient;

τ' = 1,0.

In the case of circular hollow structural sections with a length-to-diameter ratio (L/d) of less than 25

$$\tau = \frac{1}{\sqrt{1+\rho+\rho^2}}$$

where

$$\rho = 0,02\left(25 - L/d\right)$$

$$\tau' = 1 + \left(\frac{25\rho^2 \cdot \tau}{d/t}\right)\left(\frac{f_y}{0,68 f_{cu}}\right)$$

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where

C_p is equal to C_{rc} computed with $\phi = \phi_c$ = 1,0 and λ = 0

$$C_{ec} = \frac{\pi^2 \cdot E \cdot I_e}{(K \cdot L)^2}$$

where

$$E \cdot I_{\mathrm{e}} = E \cdot I_{\mathrm{s}} + \frac{0{,}6 E_{\mathrm{c}} \cdot I_{\mathrm{c}}}{1 + C_{\mathrm{us}}/C_{\mathrm{u}}}$$

I_{s} and I_{c} are the moment of inertia of the steel and concrete areas, respectively, as computed with respect to the centre of gravity of the cross section;

E_{c} is the modulus of elasticity of concrete as defined in 3.2;

C_{us} is the sustained axial load on the column;

C_{u} is the total axial load on the column.

18.2.3 Bending resistance

The factored bending resistance of a composite concrete-filled hollow structural section shall be taken as

$$M_{\mathrm{rc}} = C_{\mathrm{r}} \cdot e + C'_{\mathrm{r}} \cdot e'$$

where

$$e = b_{\mathrm{c}} \left(\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right)$$

$$e' = b_{\mathrm{c}} \left(\frac{1}{(2\pi - \beta)} + \frac{b_{\mathrm{c}}^2}{1{,}5\beta \cdot D^2 - 6 b_{\mathrm{c}} (0{,}5D - a)} \right)$$

where

β is in radians and found from the recursive equation:

$$\beta = \frac{\phi \cdot A_{\mathrm{s}} \cdot f_{\mathrm{y}} + 0{,}20\, \phi_{\mathrm{c}} \cdot D^2 \cdot f_{\mathrm{cu}} \left[\sin(\beta/2) - \sin^2(\beta/2) \cdot \tan(\beta/4) \right]}{\left(0{,}10 \phi_{\mathrm{c}} \cdot D^2 \cdot f_{\mathrm{cu}} + \phi \cdot D \cdot t \cdot f_{\mathrm{y}} \right)}$$

$$b_{\mathrm{c}} = \mathrm{D} \sin \left(\beta/2 \right)$$

$$a = \frac{b_{\mathrm{c}}}{2} \tan \left(\frac{\beta}{4} \right)$$

a) for a rectangular hollow structural section

$$C_{\mathrm{r}} = \frac{\phi \cdot A_{\mathrm{s}} \cdot f_{\mathrm{y}} - C'_{\mathrm{r}}}{2}$$

where

$$C'_r = 0{,}8\,\phi_c \cdot a \cdot f_{cu}(b - 2t)$$

$$C_r + C'_r = T_r = \phi \cdot A_{st} \cdot f_y$$

NOTE The concrete in compression is taken to have a rectangular stress block of intensity $0{,}8f_{cu}$ over a depth of $a = 0{,}85c$ where c is the depth of concrete in compression.

b) for a circular hollow structural section

$$C_r = \phi \cdot f_y \cdot \beta \frac{D \cdot t}{2};$$

$$C'_r = 0{,}8\,\phi_c \cdot f_{cu}\left[\frac{\beta \cdot D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right];$$

where

conservatively, M_{rc} may be taken as

$$M_{rc} = (Z - 2t \cdot h_n^2)\,\phi \cdot f_y + 0{,}80\left[\frac{2}{3}(0{,}5D - t)^3 - (0{,}5D - t)h_n^2\right]\phi_c \cdot f_{cu}$$

where

$$h_n = \frac{\phi_c \cdot A_c \cdot f_{cu}}{2D \cdot \phi_c \cdot f_{cu} + 4t(2{,}5\phi \cdot f_y - \phi_c \cdot f_{cu})}$$

Z is the plastic modulus of the steel section alone.

18.2.4 Axial compression and bending

Composite concrete filled hollow structural sections required to resist both bending moments and axial compression shall be proportioned analogously with 13.8.2 so that

$$\frac{C_u}{C_{rc}} + \frac{B \cdot \omega_1 \cdot M_u}{M_{rc}\left(1 - \dfrac{C_u}{C_{ec}}\right)} \le 1{,}0 \text{, and}$$

$$\frac{M_u}{M_{rc}} \le 1{,}0$$

where

M_{rc} is defined in 18.2.3;

$$B = \frac{C_{rco} - C_{rcm}}{C_{rco}};$$

C_{rco} is the factored compressive resistance with $\lambda = 0$;

$C_{rcm} = 0{,}8\phi_c \cdot A_c \cdot f_{cu}$.

18.3 Partially-encased composite columns

Design of partially-encased composite columns shall be in accordance with the CSA S16.

18.4 Encased composite columns

18.4.1 General

The provisions of this clause apply to doubly-symmetrical steel columns encased in concrete.

The design procedure applies within the following limitations:

a) the steel section is a class 1, 2 or 3 section;

b) $A_s \geq 0{,}04$ of the gross cross-sectional area;

c) $A_s + A_r \leq 0{,}20$ of the gross cross-sectional area;

d) concrete is of normal density and has a compressive strength, f_{cu}, between 25 MPa and 70 MPa;

e) the specified yield strength of structural steel, f_y, does not exceed 355 MPa; and

f) the specified yield strength of reinforcement, f_{yr}, does not exceed 400 MPa.

18.4.2 Compressive resistance

The factored compressive resistance of a steel concrete encased composite column shall be taken as

$$C_{rc} = (\phi \cdot A_s \cdot f_y + 0{,}68\phi_c \cdot A_c \cdot f_{cu} + \phi_r \cdot A_r \cdot f_{yr})(1 + \lambda^{2n})^{-1/n}$$

where

A_r is the area of longitudinal reinforcement;

n is 1,34;

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}};$$

where

C_p is equal to C_{rc} computed with ϕ, ϕ_c and ϕ_r = 1,0 and $\lambda = 0$;

C_{ec} is as defined in 18.2.2.

18.4.3 Reinforcement

18.4.3.1 Concrete encasement of a steel core shall be reinforced with longitudinal bars, and lateral ties which extend completely around the structural steel core. The encasement shall provide at least 40 mm of clear cover outside of both transverse and longitudinal reinforcement. The longitudinal bars shall meet the following requirements:

a) load carrying bars shall be continuous at framed levels;

b) the area shall be not less than 0,01 times the total gross cross-sectional area; and

c) a vertical bar shall be located at every corner; other longitudinal bars shall be spaced not further than one-half of the least side dimension of the composite section.

18.4.3.2 The lateral ties shall meet the following requirements:

a) R16 ties shall be used except that R10 ties may be used when no side of the composite section exceeds 500 mm; and

b) the vertical spacing shall not exceed the lesser of two-thirds of the least side dimension of the cross section, 16 diameters of the longitudinal bars, or 500 mm.

18.4.4 Columns with multiple steel sections

Where the composite cross-section includes two or more steel sections, the steel sections shall be considered, before hardening of the concrete, as built-up members in accordance with the requirements of clause 19.

18.4.5 Load transfer

The portion of the total axial load resisted by the concrete shall be developed by direct bearing at connections. The bearing strength of concrete may be taken as 1,35 $\phi_c \cdot f_{cu} \cdot A_L$ where A_L is the loaded area, provided that the concrete is restrained against lateral expansion.

18.4.6 Bending resistance

For the bending resistance of encased composite columns, refer to the *Guide to Stability Design Criteria for Metal Structures*.

19 General requirements for built-up members

19.1 Members in compression

19.1.1 All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of clauses 10 and 11.

19.1.2 All component parts that are in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four hole diameters apart for a distance equal to 1,5 times the width of the member or by continuous welds having a length not less than the width of the member.

19.1.3 Unless closer spacing is required for transfer of load or for sealing inaccessible surfaces, the longitudinal spacing in-line between intermediate bolts or the clear longitudinal spacing between intermittent welds for the outside plate component of built-up compression members shall not exceed the following:

a) when the bolts or intermittent welds are staggered on adjacent lines, $525t/\sqrt{f_y}$ mm, but not more than 450 mm; and

b) when the bolts on all gauge lines or intermittent welds along the component edges are not staggered, $330t/\sqrt{f_y}$ mm, but not more than 300 mm

where

t = thickness of the outside plate.

19.1.4 Compression members composed of two or more sections in contact or separated from one another shall be interconnected in such a way that the slenderness ratio of any component, based on its least radius of gyration and the distance between interconnections, shall not exceed that of the built-up member. The compressive resistance of the built-up member shall be based on

a) the slenderness ratio of the built-up member with respect to the appropriate axis when the buckling mode does not involve relative deformation that produces shear forces in the interconnectors,

b) an equivalent slenderness ratio, ρ_e with respect to the axis orthogonal to that in (a) above, when the buckling mode involves relative deformation that produces shear forces in the interconnectors, taken as

$$\rho_e = \sqrt{\rho_0^2 + \rho_i^2}$$

where

ρ_e is the equivalent slenderness ratio of the built-up member;

ρ_0 is the slenderness ratio of the built-up member acting as an integral unit;

ρ_i is the maximum slenderness ratio of the component part of a built-up member between interconnectors.

For built-up members composed of two interconnected sections in contact, or separated only by filler plates, such as back-to-back angles or channels, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1,0 when the fasteners are snug-tight bolts, and 0,65 when welds or friction-grip bolts are used.

For built-up members composed of two interconnected sections separated by lacing or batten plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1,0 for both snug-tight and friction-grip bolts and for welds.

19.1.5 For starred angle compression members interconnected at least at the one-third points, the requirements of 19.1.4 need not apply.

19.1.6 The fasteners and interconnecting parts, if any, of members defined in 19.1.4 shall be proportioned to resist a force equal to 0,01 times the total force in the built-up member.

19.1.7 The spacing requirements of 19.1.3, 19.2.3 and 19.2.4 may not always provide a continuous tight fit between components in contact. When the environment is such that corrosion may cause a serious problem, the spacing of bolts or welds may need to be less than the specified maximum.

19.1.8 Open sides of compression members built up from plates or sections shall be connected to each other by lacing, batten plates or perforated cover plates.

19.1.9 Lacing shall provide a complete triangulated shear system and may consist of bars, rods or sections. Lacing shall be proportioned to resist a shear normal to the longitudinal axis of the member of not less than 0,025 times the total axial load on the member plus the shear from transverse loads, if any.

19.1.10 The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be 0,50 times that distance.

19.1.11 Lacing members shall preferably be inclined to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.1.12 Lacing systems shall have battens in the plane of the lacing and as near to the ends as practicable, and at intermediate points where lacing is interrupted. Such battens may be plates (tie plates) or sections.

19.1.13 End tie plates used as battens shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall have a length of not less than one-half of that prescribed for end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components. The longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component or, alternatively, a total length of weld not less than one-third the length of tie plate shall be used.

19.1.14 Sections used as battens shall be proportioned and connected to transmit from one main component to the other a longitudinal shear equal to 0,05 times the axial compression in the member.

19.1.15 Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compressive members. The net width of such plates at access holes shall be assumed to be available to resist axial load, provided that

a) the width-to-thickness ratio complies with clause 11,

b) the length of the access hole does not exceed twice its width,

c) the clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member, and

d) the periphery of the access hole at all points has a minimum radius of 40 mm.

19.1.16 Battens consisting of plates or sections may be used on open sides of built-up compression members that do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length, and elsewhere as determined by 19.1.4.

19.1.17 Battens shall have a length of not less than the distance between the lines of the bolts or welds connecting them to the main components of the member, and shall have a thickness of not less than 1/60 of this distance, if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist, simultaneously, a longitudinal shear force

$$V_u = \frac{0{,}025\, C_u \cdot d}{n \cdot a}$$

and a moment,

$$M_u = \frac{0{,}025\, C_u \cdot d}{2n}$$

where

d is the longitudinal centre-to-centre distance between battens, in millimetres;

a is the distance between lines of bolts or welds connecting the batten to each main component, in millimetres;

n is the number of parallel planes of battens.

19.2 Members in tension

19.2.1 Members in tension composed of two or more sections, plates or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.2.2 Members in tension composed of two plate components in contact, or a section and a plate component in contact, shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed 36 times the thickness of the thinner plate, nor 450 mm (see 19.1.3).

19.2.3 Members in tension composed of two or more sections in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see 19.1.3).

19.2.4 Members in tension composed of two separated main components may have either perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length of not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall comply with the requirements of 19.1.15(b), (c), and (d).

19.3 Open box-type beams and grillages

Two or more rolled beams or channels used side-by-side to form a flexural member shall be connected together at intervals of not more than 1 500 mm. Through-bolts and separators may be used provided that, in beams having a depth of 300 mm or more, no fewer than two bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other or distributed between the beams, battens having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and coating.

20 Plate walls

Design of plate wall columns shall be in accordance with the CSA S16.

21 Connections

21.1 Alignment of members

The centroidal axes of axially loaded members that meet at a joint shall intersect at a common point if practicable; alternately, the results of bending due to the joint eccentricity shall be provided for.

21.2 Unrestrained members

Except as otherwise indicated in the structural design documents as a rule, all connections of beams, girders and trusses shall be designed and detailed as flexible and as a rule may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 Restrained members

When beams, girders or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load.

When beams are rigidly framed to the flange of an I-section or H-section column, stiffeners shall be provided on the column web if the following bearing and tensile resistances of the column flange are exceeded:

a) opposite the compression flange of the beam when

$$B_r = \phi_{bi} \cdot t_{wc} \left(t_{fb} + 10 t_{fc} \right) f_{yc} < \frac{M_u}{h_b}$$

where

f_{yc} is the specified yield point of the column;

t_{fb} is the thickness of the beam flange;

t_{fc} is the thickness of the column flange;

h_b is the depth of the beam;

t_{wc} is the thickness of the column web.

Except that for members with class 3 or 4 webs.

$$B_r = \phi_{bi} \frac{640\,000\, t_{wc} \left(t_{fb} + 10\, t_{fc} \right)}{\left(h_{wc} / t_{wc} \right)^2}$$

where

h_{wc} is the clear depth of the column web;

b) opposite the tension flange of the beam when

$$T_r = 7\,\phi \cdot t_{fc}^2 \cdot f_{yc} \; < \; \frac{M_u}{h_b}$$

The stiffener or pair of stiffeners opposite either beam flange shall develop a force equal to

$$F_{st} = \left(M_u / h_b \right) - B_r$$

Stiffeners shall also be provided on the web of columns, beams or girders if V_r calculated from 13.4.1.2 is exceeded, in which case the stiffener or stiffeners must transfer a shear force, V'_{st} , equal to

$$V'_{st} = V_u - 0{,}55\phi \cdot t_w \cdot h \cdot f_y$$

In all cases, the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of the column only, the stiffeners need not be longer than one-half of the depth of the column. When an axial tension or compression force is acting on the beam, their effects (additive only) shall be considered in the design of the stiffeners.

21.4 Connections of tension or compression members

The connections at ends of compression members not finished to bear, or in tension members, shall be designed for the full ultimate loads.

21.5 Bearing joints in compression members

Where columns or other compression members bear on bearing plates or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place to provide a satisfactory level of structural integrity. The flanges of single web members shall be connected.

21.6 Lamellar tearing

The details of corner-joints or T-joints of rolled structural members or plates involving transfer of tensile forces in the through-thickness direction, resulting from shrinkage due to welding executed under conditions of restraint, shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to address the possibility of lamellar tearing.

21.7 Placement of fasteners and welds

Except in members subject to fatigue (see clause 26), positioning fillet welds to balance the forces about the neutral axis or axes for end connections of single-angle, double-angle or similar types of axially loaded members is not required. Eccentricity between the centroidal axes of such members and the gauge lines of bolted end connections may also be neglected. In axially loaded members subject to fatigue, the fasteners or welds in end connections shall have their centroid on the centroidal axis of the member unless provision is made for the effect of the resulting eccentricity.

21.8 Packings

21.8.1 Packings in bolted connections

When load-carrying fasteners pass through packings with a total thickness greater than 6 mm, the packings shall be extended beyond the splice material and the packing extension shall be secured by sufficient fasteners to distribute the total force in the connected element uniformly over the combined cross-section of the connected element and the packing. Alternatively, an equivalent number of fasteners shall be included in the connection.

21.8.2 Packings in welded connections

In welded construction, any packing with a total thickness greater than 6 mm shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the packing, as an eccentric load. Welds that connect the splice plate to the packing shall be of sufficient strength to transmit the splice plate load and shall be long enough to avoid overloading the packing along the toe of the weld. Any packing that is 6 mm or less in thickness shall have its edges made flush with the edges of the splice plate and the required weld size shall be equal to the thickness of the packing plate plus the size necessary to transmit the splice plate load.

21.9 Welds in combination

If two or more of the general types of weld (groove, fillet, plug or slot welds) are combined in a single connection, the effective capacity of each shall be calculated separately with reference to the axis of the group in order to determine the factored resistance of the combination.

21.10 Fasteners and welds in combination

21.10.1 New connections

When approved by the designer, high-strength bolts in friction-grip connections can be considered as sharing the specified load with welds in new work, provided that the factored resistance of either the high-strength bolts, or the welds, is equal to, or greater than, the effect of the ultimate loads. At the specified load level, the load sharing shall be on the basis of the proportional capacities of the bolts in the friction-grip connection and 0,70 times the factored resistance of the welds.

21.10.2 Existing connections

In making alterations to structures, existing rivets and friction-grip bolts shall be used to carry forces resulting from existing dead loads and welding shall be proportioned to carry all additional loads.

21.11 High-strength friction-grip bolts and rivets in combination

In making alterations, rivets and friction-grip bolts shall be considered as sharing forces due to specified dead and live loads.

22 Design and detailing of bolted connections

22.1 General

This clause deals primarily with class 4.8, 8.8, 8.8S, 10.9 and 10.9S bolt assemblies complying with the relevant parts of SANS 1700, or equivalent fasteners. The bolts might or might not be required to be installed to a specified minimum tension, depending on the type of connection.

NOTE The term "class" is used here to indicate the material of which a bolt is made, in accordance with SANS 1700.

22.2 Design of bolted connections

22.2.1 Use of snug-tightened high-strength bolts

Snug-tightened high-strength bolts shall be used in all connections except those specified in 22.2.2.

22.2.2 Use of high-strength friction-grip bolts

Pre-tensioned high-strength friction-grip bolts (class 8.8S or 10.9S) shall be used in

a) friction-grip connections where slippage cannot be tolerated (such connections include those subject to fatigue or to frequent load reversal, or those in structures sensitive to deflection),

b) shear connections proportioned in accordance with seismic requirements,

c) all elements resisting crane loads,

d) connections subject to impact or cyclic loading, and

e) connections where the bolts are subject to cyclic tensile loading (see 13.12.2.3).

22.2.3 Joints subject to fatigue loading

Joints subject to fatigue loading shall be proportioned in accordance with clause 26.

22.2.4 Effective bearing area

The effective bearing area of bolts shall be the nominal diameter multiplied by the length in bearing. For countersunk bolts, half of the depth of the countersink shall be deducted from the bearing length.

22.2.5 Fastener components

22.2.5.1 Structural bolt assemblies

Except as provided in 22.2.5.3, bolts, nuts and washers for structural bolt assemblies shall comply with SANS 1700.

22.2.5.2 Galvanized bolt assemblies

Galvanized bolt assemblies shall comply with SANS 1700.

22.2.5.3 Alternatives to SANS 1700 bolt assemblies

Other fasteners that comply with SANS 1700 may be used. The dimensions of such fasteners may differ and their use shall be subject to the approval of the designer.

22.3 Detailing of bolted connections

22.3.1 Minimum pitch

The minimum distance between centres of bolt holes shall be 2,7 times the bolt diameter.

22.3.2 Minimum edge distance

The minimum distance from the centre of a bolt hole to any edge shall be in accordance with table 8.

Table 8 — Minimum edge distance for bolt holes

1	2	3
Bolt diameter mm	**Minimum edge distance** mm	
	At sheared edge	**At rolled, sawn or gas-cut edge**[a]
16	28	22
20	34	26
22	38	28
24	42	30
27	48	34
30	52	38
36	64	46
>36	1,75 x diameter	1,25 x diameter

[a] Gas-cut edges shall be smooth and free from notches. Edge distance in this column may be decreased by 3 mm when the hole is at a point where calculated stress under factored loads is not more than 0,3 of the yield stress.

22.3.3 Maximum edge distance

The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part, but not greater than 150 mm.

22.3.4 Minimum end distance

In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance (from the centre of the end fastener to the nearest end of the connected part) shall be governed by the edge distance values given in table 8. In members having either one or two bolts in the line of load, the end distance shall be not less than 1,5 bolt diameters.

22.3.5 Bolt holes

22.3.5.1 Holes may be punched, sub-punched or sub-drilled and reamed, or drilled, in accordance with SANS 2001:CS1. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size. Oversized or slotted holes may be used with high-strength bolts 16 mm in diameter and larger when approved by the designer.

Joints that use enlarged or slotted holes shall be proportioned in accordance with the requirements of 13.11 and 13.12 and clause 23, and shall comply with the following conditions:

a) oversize holes are 4 mm larger than bolts 22 mm and less in diameter, 6 mm larger than bolts 24 mm in diameter, and 8 mm larger than bolts 27 mm and greater in diameter. Oversize holes shall not be used in bearing-type connections but can be used in any or all plies of friction-grip connections. Hardened washers shall be used under heads or nuts adjacent to the plies containing oversize holes;

b) short slotted holes are 2 mm wider than the bolt diameter and have a length that does not exceed the oversize diameter provisions of (a) above by more than 2 mm. They can be used in any or all plies of friction-grip or bearing-type connections. Such slots can be used without regard to the direction of loading in friction-grip connections but shall be normal to the direction of the load in bearing-type connections. For pre-tensioned bolts, hardened washers shall be used under heads or nuts adjacent to the plies containing the slotted holes;

c) long slotted holes are 2 mm wider than the bolt diameter and have a length greater than that allowed in (b) above but not more than 2,5 times the bolt diameter and shall be used

   i) in friction-grip connections without regard to the direction of loading. Slip resistance shall be decreased in accordance with 13.12,

   ii) in bearing-type connections with the long dimension of the slot normal to the direction of loading,

   iii) in only one of the connected parts at an individual faying surface of either a friction-grip or bearing-type connection, and

   iv) provided that structural plate washers or a continuous bar not less than 8 mm in thickness cover long slots that are in the outer plies of joints. These washers or bars shall have a size sufficient to completely cover the slot after installation;

d) when pre-tensioned class 10.9S bolts greater than 24 mm in diameter are used in oversize or slotted holes, hardened washers shall be at least 8 mm in thickness.

22.3.5.2 The maximum and minimum edge distance for bolts in slotted or oversize holes (as permitted in 22.3.5.1) shall comply with the requirements given in 22.3.2, 22.3.3, and 22.3.4, assuming that the fastener can be placed at any extremity of the slot or hole.

23 Installation and inspection of bolted joints

The installation and inspection of bolted joints shall be in accordance with SANS 2001:CS1.

24 Welding

24.1 Design of arc-welded joints

Welded joints shall be designed in accordance with

a) 13.13 for factored resistance to static loading; and

b) clause 26 for resistance to fatigue loading.

For all other requirements, AWS D1.1 shall be complied with.

24.2 Design of resistance-welded joints

The resistance of resistance-welded joints shall be in accordance with SANS 10162-2.

24.3 Welding

Welding shall be done in accordance with SANS 2001:CS1.

25 Column bases and holding-down bolts

25.1 Loads

Suitable provision shall be made to transfer ultimate axial loads, including uplift, shears and moments to footings and foundations. Forces present during construction as well as those present in the finished structure shall be resisted.

25.2 Resistance

25.2.1 Concrete in compression

The compressive resistance of concrete shall be determined in accordance with SANS 10100-1. When compression exists over the entire base plate area, the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by the depth minus 2*e*, where *e* is the eccentricity of the column load.

25.2.2 Tension

25.2.2.1 Holding-down bolts

The factored tensile resistance of a holding-down bolt shall be taken as

$$T_r = \phi_b \cdot A_n \cdot f_u$$

where

ϕ_b = 0,67;

A_n is the tensile area of the bolts;

$$= \frac{\pi}{4}(d - 0{,}938P)^2$$

where

P is the pitch of thread, in millimetres;

d is nominal thread diameter.

25.2.2.2 Pull-out

For the requirements for transferring the tensile forces from the anchors to the concrete, see SANS 10100-1. Full anchorage is obtained when the factored pull-out resistance of the concrete is equal to, or greater than, the factored tensile resistance of the bolts.

25.2.3 Shear

25.2.3.1 Shear transfer mechanisms

Shear resistance may be developed by friction between the base plate and the foundation unit or by bearing of the holding-down bolts or shear lugs against the concrete.

25.2.3.2 Holding-down bolts in bearing

The factored bearing resistance of a holding-down bolt shall be taken as

$$B_r = 1{,}12\ \phi_c \cdot A \cdot f_{cu}$$

where

$\phi_c = 0{,}60$;

A is the bearing area, taken as the product of the rod diameter, d, and an assumed depth of $5d$.

25.2.3.3 Holding-down bolts in shear

The factored shear resistance of a holding-down bolt shall be taken as

$$V_r = 0{,}60\ \phi_b \cdot A_{hd} \cdot f_u$$

where

A_{hd} is the cross-sectional area of a holding-down bolt based on its nominal diameter of the threaded portion.

When the bolt threads are intercepted by the shear plane, the factored shear resistance shall be taken as 0,70 V_r.

25.2.4 Holding-down bolts in shear and tension

A holding-down bolt required to develop resistance to both tension and shear, shall be proportioned so that

$$(V_u/V_r) + (T_u/T_r) \leq 1{,}4$$

where

V_u is the portion of the total shear per rod transmitted by bearing of the holding-down bolts on the concrete.

25.2.5 Holding-down bolts in tension and bending

A holding-down bolt required to develop resistance to both tension and bending shall be proportioned to comply with the requirements of 13.9(a). The tensile and moment resistances, T_r and M_r, shall be based on the properties of the cross-section at the critical section; M_r shall be taken as $\phi_b \cdot Z_{pl} \cdot f_y$.

25.2.6 Moment on column base

Moment resistance of a column base shall be taken as the couple formed by the tensile resistance determined in accordance with 25.2.2 and by the concrete compressive resistance determined in accordance with 25.2.1.

26 Fatigue

26.1 General

In addition to complying with the requirements of clause 26 for fatigue, any member or connection shall also comply with the requirements for the static load conditions using the factored loads. Specified loads shall be used for all fatigue calculations. A specified load less than the maximum specified load, but acting with a greater number of cycles, may govern and shall be considered. Members and connections subjected to fatigue loading shall be designed, detailed and fabricated so as to minimize stress concentrations and abrupt changes in cross-section. The life of the structure shall be taken as 50 years, unless otherwise stated.

26.2 Proportioning

In the absence of more specific information, which is subject to the approval of the owner, the requirements of clause 26 in its entirety provide guidance in proportioning members and parts. Fatigue resistance shall be provided only for those loads considered to be repetitive.

26.3 Live load-induced fatigue

26.3.1 Calculation of stress range

The controlling stress feature in load-induced fatigue is the range of stress to which the element is subjected. This is calculated using ordinary elastic analysis and the principles of mechanics of materials. More sophisticated analysis is required only in cases not covered in table 10 and figure 2, for example, major access holes and cut-outs. Stress range is the algebraic difference between the maximum stress and minimum stress at a given location: thus, only the stress due to live load need be calculated.

The load-induced fatigue provisions need be applied only at locations that undergo a net applied tensile stress. Stress ranges that are completely in compression need not be investigated for fatigue.

26.3.2 Design criteria

For load-induced fatigue

$$f_{fr} \geq f_{sr}$$

where

f_{fr} is the fatigue resistance;

f_{sr} is the calculated stress range at the detail due to application of the moving load;

$$= \left(\frac{\gamma}{n \cdot N} \right)^{1/3} \geq f_{srt}$$

where

γ is the fatigue life constant, see 26.3.3;

n is the number of stress range cycles at given detail for each application of the moving load;

N is the number of applications of the load;

f_{srt} is the constant amplitude threshold stress range, see 26.3.3 and 26.3.4.

26.3.3 Cumulative fatigue damage

The total damage that results from fatigue loading, not applied at constant amplitude, shall satisfy

$$\sum\left[\frac{(n \cdot N)_i}{N_{fi}}\right] \leq 1{,}0$$

where

$(n{\cdot}N)_i$ is the number of expected stress range cycles at stress range level i;

N_{fi} is the number of cycles that would cause failure at stress range level i.

The summation shall include low stress cycles with stress ranges less than f_{srt} beyond the limit from 26.3.2 where $f_{sr} = f_{srt}$, on the basis that for such cycles the fatigue resistance $f_{sr}^{'}$ shall be taken as

$$f_{sr}^{'} = \left(\frac{\gamma'}{n \cdot N}\right)^{1/5}$$

where

' is obtained by setting $\left(\frac{\gamma}{n \cdot N'}\right)^{1/3} = \left(\frac{\gamma'}{n \cdot N'}\right)^{1/5}$ for the value of $n{\cdot}N = n{\cdot}N'$ at which $f_{sr} = f_{srt}$

where

N' is the number of passages of a moving vehicle at which $f_{sr} = f_{srt}$.

26.3.4 Fatigue constants and detail categories

The fatigue constants, γ, γ', $n{\cdot}N'$ and f_{srt} are given in table 9 and figure 1. The detail categories shall be obtained from table 10 and figure 2.

For high-strength bolts, see also 13.12.1.3.

26.3.5 Limited number of cycles

Except for fatigue sensitive details with high stress ranges (probably with stress reversal), no special considerations beyond those given in 26.1 need apply in the event that the number of stress range cycles, $n{\cdot}N$, over the life of the structure, expected to be applied at the given detail, is less than the greater of

γ / f_{sr}^3 and 20 000.

26.4 Distortion-induced fatigue

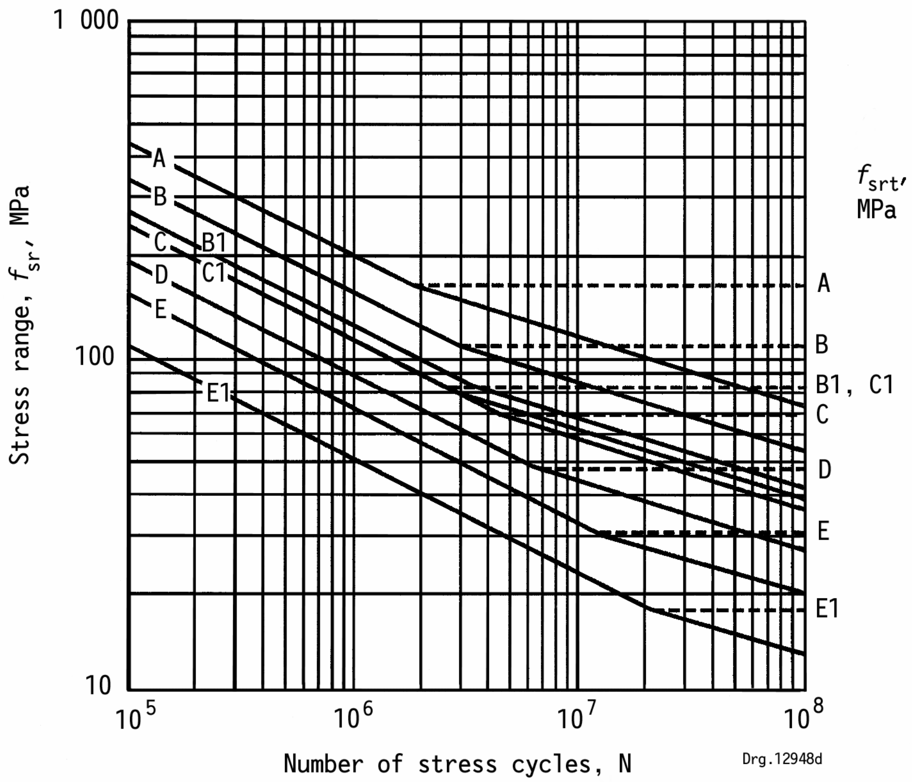
26.4.1 General

Members and connections shall be detailed to reduce distortion-induced fatigue that can occur in regions of high strain at the interconnection of members undergoing differential displacements. Whenever practicable, all components that make up the cross-section of the primary member shall be fastened to the interconnection member.

26.4.2 Plate girders with $h_w / t_w > 3150 / \sqrt{f_y}$ shall not be used under fatigue conditions.

Table 9 — Fatigue constant for various detail categories

1	2	3	4	5
Detail category	**Fatigue life constant** γ MPa	**Constant amplitude threshold stress range** f_{srt} MPa	**Cycles n·N′**	**Fatigue life constant** γ', MPa
A	$8\,190 \times 10^9$	165	$1{,}82 \times 10^6$	223×10^{15}
B	$3\,930 \times 10^9$	110	$2{,}95 \times 10^6$	$47{,}6 \times 10^{15}$
B1	$2\,000 \times 10^9$	83	$3{,}50 \times 10^6$	$13{,}8 \times 10^{15}$
C	$1\,440 \times 10^9$	69	$4{,}38 \times 10^6$	$6{,}86 \times 10^{15}$
C1	$1\,440 \times 10^9$	83	$2{,}52 \times 10^6$	$9{,}92 \times 10^{15}$
D	721×10^9	48	$6{,}52 \times 10^6$	$1{,}66 \times 10^{15}$
E	361×10^9	31	$12{,}1 \times 10^6$	$0{,}347 \times 10^{15}$
E1	128×10^9	18	$22{,}0 \times 10^6$	$0{,}0415 \times 10^{15}$



NOTE Dotted lines show values of f_{srt}.

Figure 1 — Fatigue constants for various detail categories A to E1 (see clause 26.3.4)

Table 10 — Detail categories for load-induced fatigue

1	2	3	4
General condition	**Situation**	**Detail category**	**Illustrative example, see figure 2**
Plain members	Base metal:	A	1,2
	• with rolled or cleaned surfaces. Flame-cut edges with a surface roughness not exceeding 1000 (25 μm) as defined in CSA B95		
	• of unpainted weathering steel	B	
	• at re-entrant corners of geometric discontinuities such as copes, cuts or block-outs	B	2a
	• at net section of eyebar heads and pin plates	E	
Built-up members	Base metal and weld metal in components, without attachments, connected by:		3, 4, 5, 7
	• continuous full-penetration groove welds with backing bars removed, or	B	
	• continuous fillet welds parallel to the direction of applied stress	B	
	• continuous full-penetration groove welds with backing bars in place, or	B1	
	• continuous partial-penetration groove welds parallel to the direction of applied stress	B1	
	Base metal at ends of partial-length cover plates:		
	• bolts in friction grip	B	22
	• narrower than the flange, with or without end welds, or wider than the flange with end welds		
	• flange thickness ≤ 20 mm	E	7
	• flange thickness > 20 mm	E1	
	• wider than the flange without end welds	E1	
Groove-welded splice connections with weld soundness established by non destructive testing and all required grinding in the direction of the applied stresses	Base metal and weld metal at full-penetration groove-welded splices:		
	• of plates of similar cross-sections with welds ground flush	B	8, 9
	• with 600 mm radius transitions in width with welds ground flush	B	11
	• with transitions in width or thickness with welds ground to provide slopes not steeper than 1,0 to 2,5		10, 10A
	• G40.21-700Q and 700QT base metal	B1	
	• other base metal grades	B	
	• with or without transitions having slopes not greater than 1,0 to 2,5, when weld reinforcement is not removed	C	8, 9, 10, 10A
	• at weld access holes		
	• of rolled members	C	
	• of build-up members	D	

Table 10 *(continued)*

1	2	3	4
General condition	**Situation**	**Detail category**	**Illustrative example, see figure 2**
Longitudinally loaded groove-welded attachments	Base metal at details attached by full- or partial-penetration groove welds:		
	• when the detail length in the direction of applied stress is:		
	• less than 50 mm	C	6, 18
	• between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18
	• greater than either 12 times the detail thickness or 100 mm		
	• detail thickness < 25 mm	E	18
	• detail thickness $\geq$ 25 mm	E1	18
	• with a transition radius, *R*, with the end welds ground smooth, regardless of detail length:		12
	• $R \geq$ 600 mm	B	
	• 600 mm > $R \geq$ 150 mm	C	
	• 150 mm > $R \geq$ 50 mm	D	
	• R < 50 mm	E	
	• with a transition radius with end welds not ground smooth	E	12
Transversely loaded groove-welded attachments with weld soundness established by non destructive testing and all required grinding transverse to the direction of stress	Base metal at detail attached by full-penetration groove welds with a transition radius, *R*:		12
	• to flange, with equal plate thickness and weld reinforcement removed:		
	• $R \geq$ 600 mm	B	
	• 600 mm > $R \geq$ 150 mm	C	
	• 150 mm > $R \geq$ 50 mm	D	
	• R < 50 mm	E	
	• to flange, with equal plate thickness and weld reinforcement not removed, or web:		
	• > $R \geq$ 150 mm	C	
	• 150 mm > $R \geq$ 50 mm	D	
	• R < 50 mm	E	
	• to flange, with unequal plate thickness and weld reinforcement removed:		
	• $R \geq$ 50 mm	D	
	• R < 50 mm	E	
	• to flange, for any transition radius with unequal plate thickness and weld reinforcement not removed	E	

Table 10 *(continued)*

1	2	3	4
General condition	**Situation**	**Detail category**	**Illustrative example, see figure 2**
Fillet-welded connections with welds normal to the direction of stress	Base metal:		
	• at details other than transverse stiffener-to-flange or transverse stiffener-to-web connections	C (see Note)	19
	• at the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C1	6
Fillet-welded connections with welds normal and/ or parallel to the direction of stress	Shear stress on weld throat	E	16
Longitudinally loaded fillet-welded attachments	Base metal at details attached by fillet welds:		
	• when the detail length in the direction of applied stress is:		
	• less than 50 mm or stud-type shear connectors	C	13, 15, 18, 20
	• between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18, 20
	• greater than either 12 times the detail thickness or 100 mm		7, 16, 18 20
	• detail thickness < 25 mm	E	
	• detail thickness ≥ 25 mm	E1	
	• with a transition radius, *R*, with the end of welds ground smooth, regardless of detail length		12
	• $R \geq 50$ mm	D	
	• $R < 50$ mm	E	
	• with a transition radius with end of welds not ground smooth	E	12
Transversely loaded fillet-welded attachments with welds parallel to the direction of primary stress	Base metal at details attached by fillet welds:		12
	• with a transition radius, *R*, with end of welds ground smooth:		
	• $R \geq 50$ mm	D	
	• $R < 50$ mm	E	
	• with any transition radius with end of welds not ground smooth	E	
Mechanically fastened connections	Base metal:		17
	• at gross section of high-strength bolted slip-critical connections, except axially loaded joints in which out-of-plane bending is induced in connected materials	B	
	• at net section of high-strength friction grip connections	B	
	• at net section of riveted connections	C	
Anchor bolts and threaded parts	Tensile stress range on the tensile stress area of the threaded part, including effects of bending	E	

Table 10 *(concluded)*

1	2	3	4
General condition	**Situation**	**Detail category**	**Illustrative example, see figure 2**
Fillet-welded hollow section to base plate	• Shear stress on fillet weld	E1	21
Class 8.8 bolts in axial tension	• Tensile stress on area A_b	See 13.12.1.3	
Class 10.9 bolts in axial tension	• Tensile stress on area A_b	See 13.12.1.3	

NOTE The fatigue resistance of fillet welds transversely loaded is a function of the effective throat and plate thickness. (Ref. Journal of the Structural Division.)

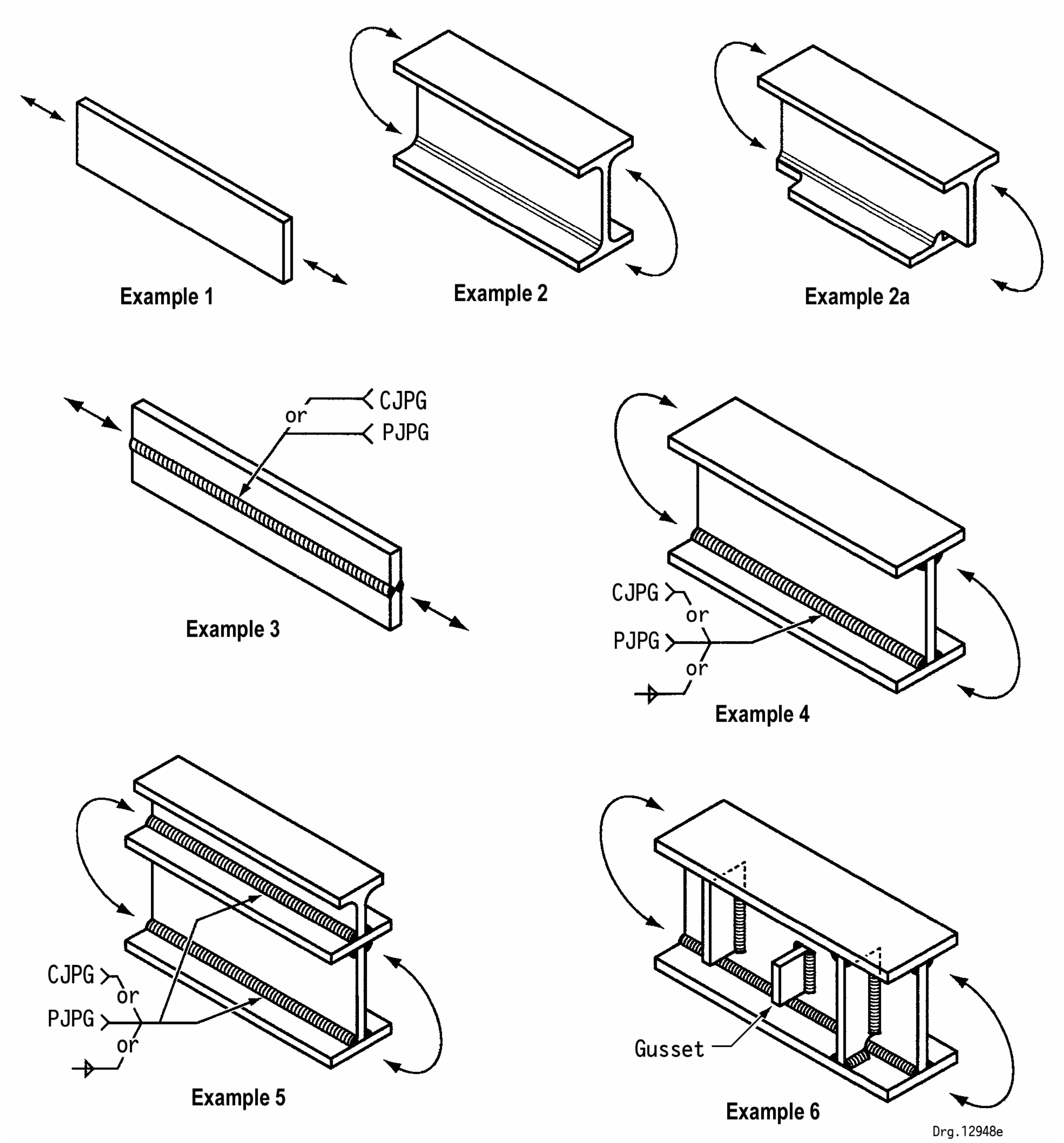
$$f_{sr} = f_{sr}^{c}\left[\left(0,06 + 0,79\, H/t_p\right)/\left(0,64\, t_p^{1/6}\right)\right]$$

where

f_{sr}^{c} is the fatigue resistance for category C as determined in accordance with 26.3.3. This assumes no penetration at the weld root;

H is the weld leg size;

t_p is the plate thickness.



NOTE The example numbers are referred to in table 10

Figure 2 — Illusrative examples of various details representing stress range categories

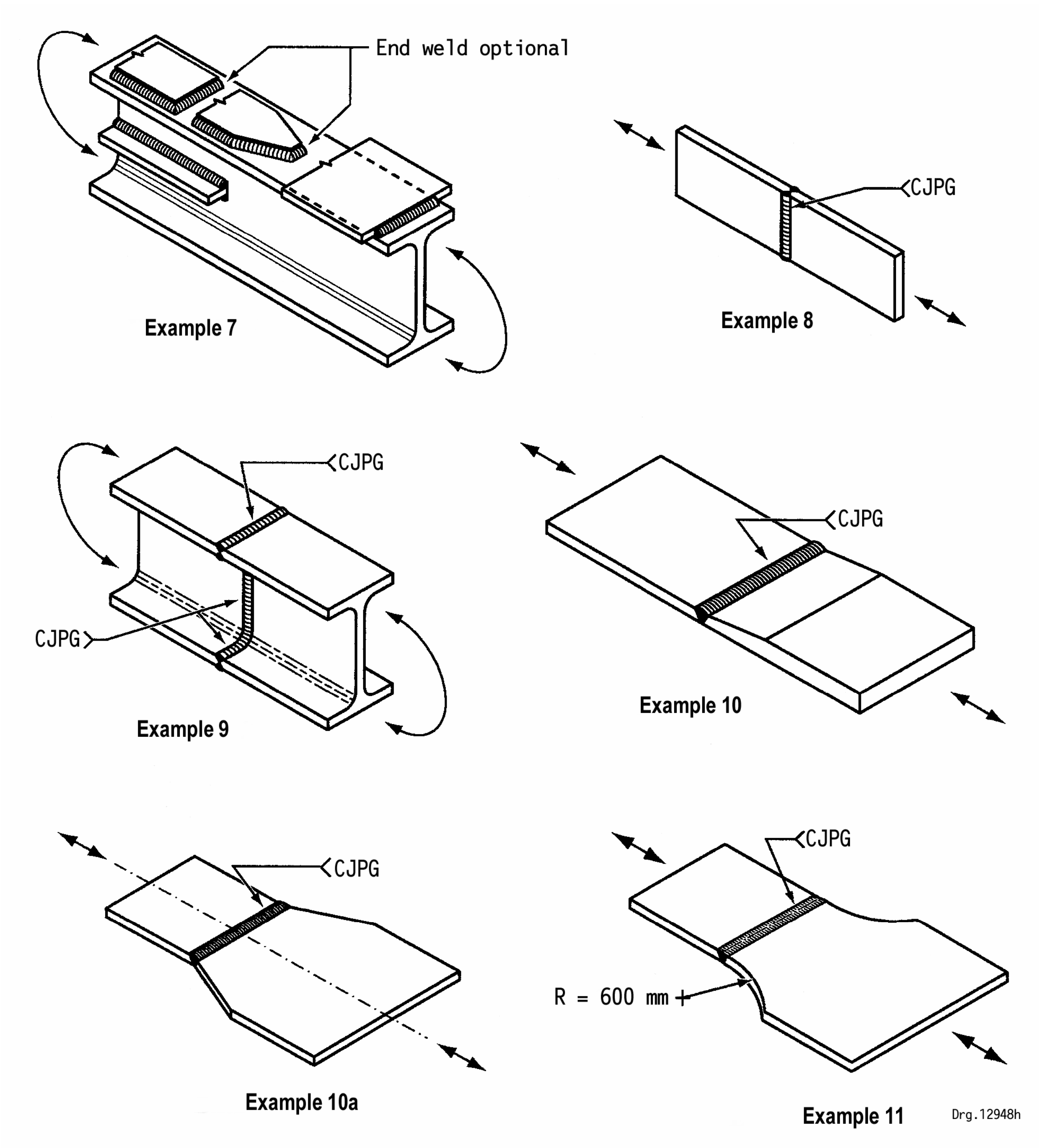
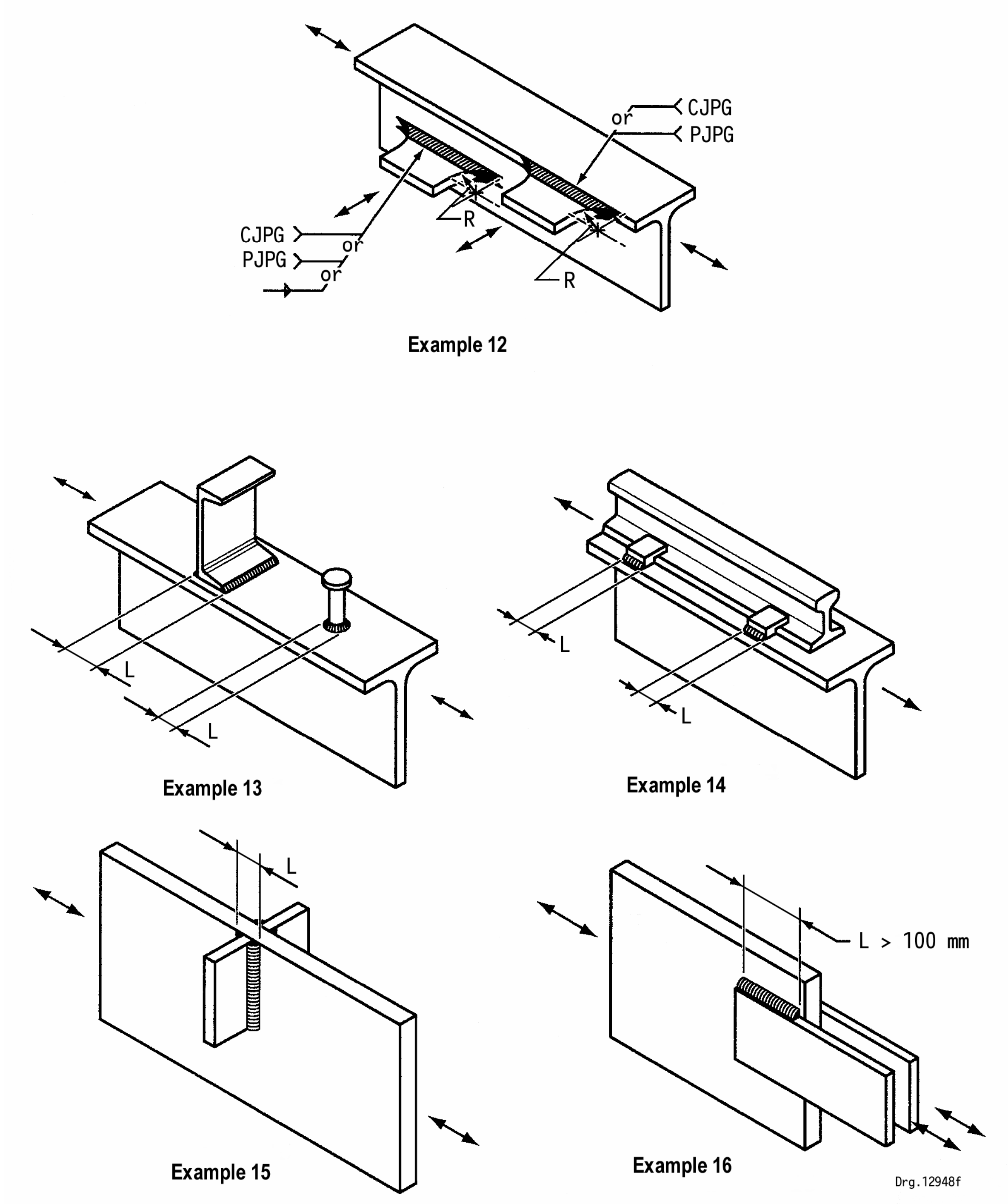


Figure 2 — *(continued)*



NOTE The example numbers are referred to in table 9.

Figure 2 — *(continued)*

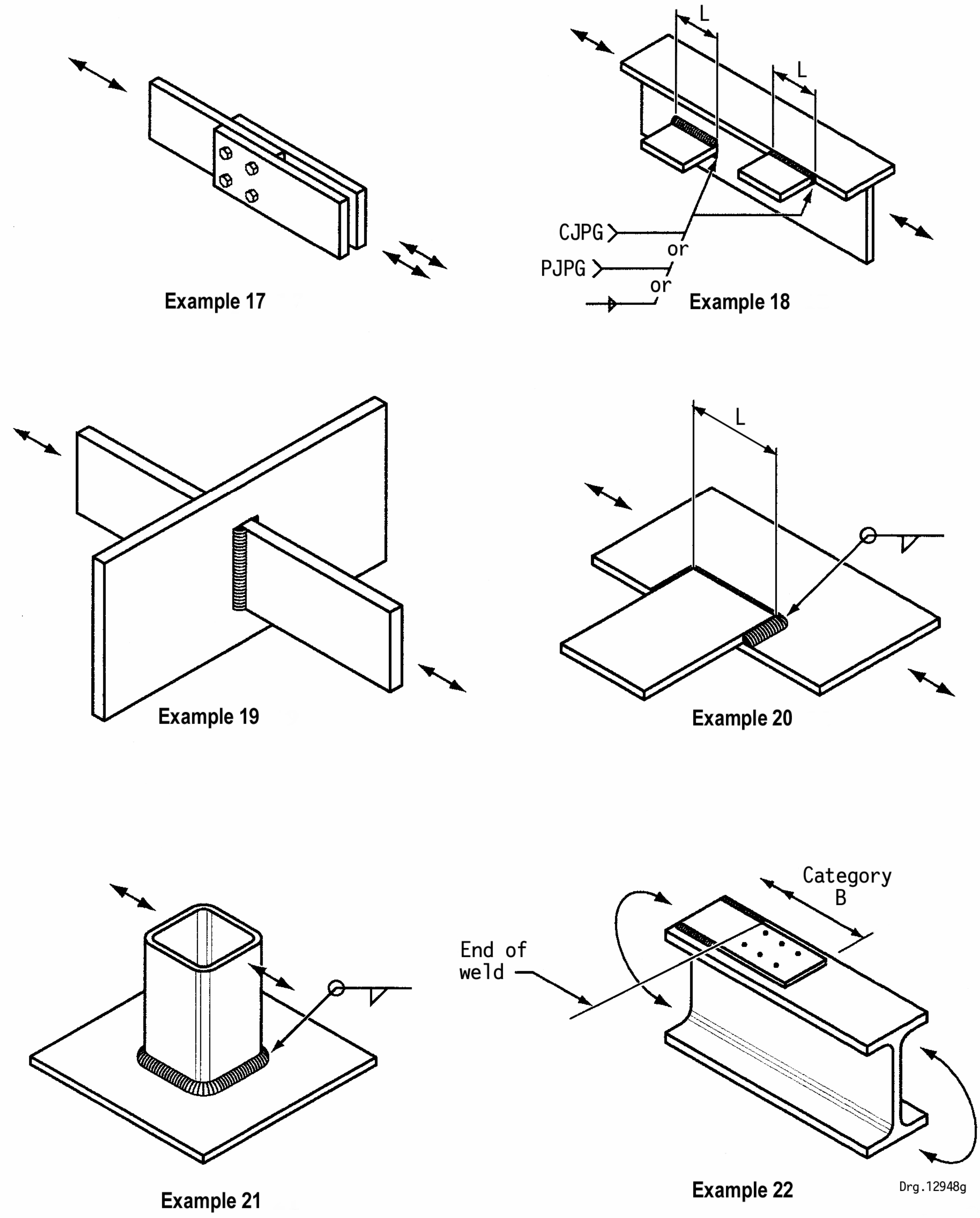


Figure 2 — *(concluded)*

27 Seismic design requirements

The provisions of this standard are adequate for the design of steel structures in South Africa and are in the seismic design requirements as specified in SANS 10160-4. **Amdt 1**

More severe seismic loading shall meet the requirements of a combination of an appropriate loading standard and a steel design standard, such as CSA S16.

28 Fabrication, erection and protection against corrosion

The fabrication and erection of structural steel shall be done in accordance with SANS 2001:CS1. No relaxation of the requirements of SANS 2001:CS1 shall be allowed where such relaxation can have an adverse effect on the strength or stability of the structure. Structures shall be adequately protected against corrosion, commensurate with the conditions to which they will be exposed during their life time (see 6.5).

Annex A
(informative)

Standard practice for structural steel

A.1 Matters concerning standard practice not covered by this standard but pertinent to the fabrication and erection of structural steel (such as a definition of structural steel items, the computation of weights, etc.) are to be in accordance with SANS 2001:CS1, unless otherwise clearly specified in the plans and standards.

Annex B
(informative)

Margins of safety

B.1 An advantage of limit-states design is that the probability of failure for different loading conditions is made more consistent by the use of distinct load factors for the different loads to which the structure is subject, than in allowable or working stress design, where a single factor of safety is used. Furthermore, different resistance factors can be applied to determine member resistances with a uniform reliability. The product of the load factor and the inverse of the resistance factor gives a number comparable to the traditional factor of safety. In this part of SANS 10162, appropriate resistance factors are defined in 13.1, 17.1.2, 17.7.1 and 18.1.1 for different limit states, structural elements and materials.

B.2 If load factors of 1,2 for permanent (self-weight) load and 1,6 for variable (live) load are used, probabilistic studies indicate that consistent probabilities of failure are determined over all ranges of permanent-to-variable load ratios. The same probabilistic studies also show that the load combination factors given in SANS 10160 (all parts) and applied only to imposed, wind or earthquake and temperature actions, and a factor of 0,90 applied to permanent load when it is counteractive to imposed loads, also result in a consistent probability of failure. **Amdt 1**

B.3 Resistance factors (see 3.1.7) generally allow for underrun in the member or connection resistance as compared to the resistance predicted. The underrun may arise from variability in material properties, dimensions and workmanship, as well as from simplifications in the mathematical derivation of the resistance equations.

For the sake of simplicity, uncertainty in the formulation of the theoretical member resistance has in some cases in this part of SANS 10162 been incorporated directly into the expression for member resistance rather than the use of a lower value for the resistance factor. This is the case for the column curve, where the curve predicting the ultimate strengths as a function of the slenderness ratio has been derived statistically, taking into account residual stresses and initial out-of-straightness.

B.4 For bolts, a resistance factor of 0,80 for bolts in shear and tension; 0,67 for bolts in bearing and 0,67 for holding-down bolts in shear, tension and bearing is used to ensure that connector failures will not occur before general failure of the member as a whole. For long bolted joints and for cases in which shear planes intersect the threads, reduction factors are applied to the resistance formulations. For welds a resistance factor of 0,67 is used.

B.5 References

Kemp, A.R., Milford, R.V. and Laurie, J.A.P., 1987. *Proposals for a comprehensive limit states formulation for South African structural standards*. The Civil Engineer in South Africa, Vol. 29, No. 9, pp. 351-360.

Annex C
(informative)

Guide for floor vibrations

C.1 General requirements

The development of floors of lighter construction, longer spans and less inherent damping may sometimes result in disturbing floor vibrations during normal human activity. The specific vibration characteristics of the floor should be evaluated by the building designer.

Such an evaluation shall, as a minimum, consider

a) the characteristics and nature of the forcing excitations,

b) the acceptance criteria for human comfort,

c) the determination of natural frequency of the floor framing systems, including the effect of continuity,

d) the modal damping ratio, and

e) the effective floor panel weights.

For information and guidance on these and other issues related to floor vibrations, see C.2.

C.2 References

AISC/CISC, 1997, Steel Design Guide Series 11. *Floor Vibrations Due to Human Activity*. American Institute of Steel Construction (AISC/CISC, 1997), Chicago.

SCI, 2000, *Design Guide for Vibrations of Long Span Composite Floors*. Document RT852. The Steel Construction Institute, Silwood Park, England.

Wyatt, T. A., 1989, *Design Guide on the Vibration of Floors*. The Steel Construction Institute, Silwood Park, England.

Annex D
(informative)

Recommended maximum values for deflections for serviceability variable and wind loads

D.1 Although the deflection criteria in table D.1 refer to serviceability variable and wind loads, the designer should consider the inclusion of serviceability permanent loads in some instances. For example, non-permanent partitions, which are classified as permanent load, should be part of the loading considered if they are likely to be applied to the structure after the completion of finishes susceptible to cracking. Because some building materials augment the rigidity provided by the steelwork, the wind load assumed to be carried by the steelwork for calculating deflections can be somewhat reduced from the design wind load used in strength and stability calculations. The more common structural elements that contribute to the stiffness of a building are masonry walls, certain types of curtain walls, masonry partitions and concrete around steel members. The maximum suggested amount of this reduction is 15%. In tall and slender structures (height greater than 4 times the width), it is recommended that the wind effects be determined by means of dynamic analysis or wind tunnel tests.

Table D.1 — Maximum deflections at serviceabiblity

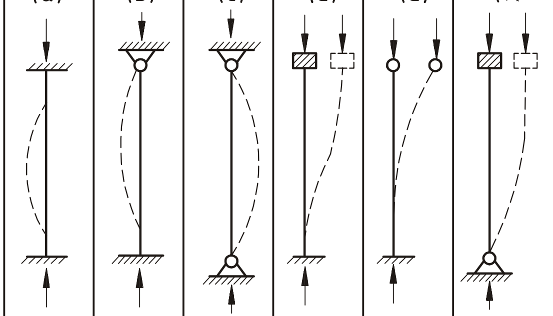
<table>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><th>Buildings</th><th>Kind deflection</th><th>Design load</th><th>Application</th><th>Maximum deflection</th></tr>
<tr><td rowspan="7">Industrial-type buildings</td><td rowspan="5">Vertical deflection</td><td>Variable load</td><td>Simple span members supporting inelastic roof coverings</td><td>1/240 of span</td></tr>
<tr><td>Variable load</td><td>Simple span members supporting elastic roof coverings</td><td>1/180 of span</td></tr>
<tr><td>Variable load</td><td>Simple span members supporting floors</td><td>1/300 of span</td></tr>
<tr><td>Maximum wheel loads (no impact)</td><td>Simple span crane runway girders for crane capacity of 225 kN and over</td><td>1/800 of span</td></tr>
<tr><td>Maximum wheel loads (no impact)</td><td>Simple span crane runway girders for crane capacity under 225 kN</td><td>1/600 of span</td></tr>
<tr><td rowspan="2">Lateral deflectio n</td><td>Crane lateral force</td><td>Simple span crane runway girders</td><td>1/600 of span</td></tr>
<tr><td>Crane lateral force or wind</td><td>Building column sway at crane level[a]</td><td>1/400 to 1/200 of height</td></tr>
<tr><td rowspan="5">All other buildings</td><td rowspan="2">Vertical deflection</td><td>Variable load</td><td>Simple span members of floors and roofs supporting construction and finishes susceptible to cracking</td><td>1/360 of span</td></tr>
<tr><td>Variable load</td><td>Simple span members of floors and roofs supporting construction and finishes not susceptible to cracking</td><td>1/300 of span</td></tr>
<tr><td rowspan="3">Lateral deflection</td><td>Wind</td><td>Building sway, due to all effects</td><td>1/400 of building height</td></tr>
<tr><td>Wind</td><td>Storey drift (relative horizontal movement of any two consecutive floors due to shear effects) in buildings in cladding and partitions without special provision to accommodate building frame deformation</td><td>1/500 of storey height</td></tr>
<tr><td>Wind</td><td>Storey drift (as above), with special provision to accommodate building frame deformation</td><td>1/400 of height</td></tr>
<tr><td colspan="5">[a] Permissible sway of industrial buildings varies considerably and depends on such factors as wall construction, building height, effect of deflection on the operation of a crane, etc.. Where the operation of the crane is sensitive to lateral deflections, a permissible lateral deflection of less than 1/400 of the height may be required.</td></tr>
</table>

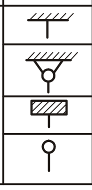
Annex E
(informative)

Effective lengths of columns

E.1 The slenderness ratio of a column is defined as the ratio of the effective length to the applicable radius of gyration. The effective length, KL, may be thought of as the actual unbraced length, L, multiplied by a factor, K, such that the product, KL, is equal to the length of a pin-ended column of equal capacity to the actual member. The effective length factor, K, of a column of finite unbraced length therefore depends on the conditions of restraint afforded to the column at its braced locations.

E.2 A variation in K between 0,65 and 2,0 would apply to the majority of cases likely to be encountered in actual structures. Figure E.1 illustrates six idealized cases in which joint rotation and translation are either fully realized or nonexistent.

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of colum is shown by dashed line						
Theoretical K value	0,5	0,7	1,0	1,0	2,0	2,0
Recommended design value when ideal conditions are approximated	0,65	0,80	1,0	1,2	2,0	2,0

End condition code		
	Rotation fixed	Translation fixed
	Rotation free	Translation fixed
	Rotation fixed	Translation free
	Rotation free	Translation free

Drg.12842

Figure E.1 — Idealized cases

Annex F
(informative)

Criteria for estimating effective column lengths in continuous frames

F.1 Because the standard requires that the in-plane behaviour of beam-columns be based on their actual lengths (provided only that, when applicable, the sway effects are included in the analysis of the structure (see 8.7)), this annex applies only to cases related to buckling; that is to axially loaded columns and beam-columns failing by out-of-plane buckling.

F.2 Figure F.1 is a nomograph applicable to cases in which the equivalent I/L of adjacent girders that are rigidly attached to the columns is known; it is based on the assumption that all columns in the portion of the framework considered reach their individual critical load simultaneously.

In the usual building frame, not all columns would be loaded so as to simultaneously reach their buckling loads; thus some conservatism is introduced in the interest of simplification.

F.3 The equation on which this nomograph is based is as follows:

$$\frac{G_U G_L}{4}\left(\pi / K\right)^2 + \frac{G_U + G_L}{2}\left(1 - \frac{\pi / K}{\tan \pi / K}\right) + 2\left[\frac{\tan \pi / 2K}{\pi / K}\right] = 1$$

F.4 Subscripts *U* and *L* refer to the joints at the two ends of the column section being considered.

G is defined as

$$G = \frac{\Sigma\left(I_c/L_c\right)}{\Sigma\left(I_g/L_g\right)}$$

where

Σ indicates a summation for all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered;

I_c is the moment of inertia about the axes perpendicular to the plane of buckling;

L_c is the unsupported length of a column section;

I_g is the moment of inertia with the axes perpendicular to the plane of buckling;

L_g is the unsupported length of a girder or other restraining member.

F.5 For column ends supported by, but not rigidly connected to, a footing or foundation, *G*, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1,0. Smaller values may be used if justified by analysis.

F.6 Refinements in girder I_g/L_g may be made when conditions at the far end of any particular girder are definitely known or when a conservative estimate can be made. For the case with no side-sway, multiply girder stiffnesses by the following factors:

a) 1,5 if the far end of the girder is pinned; and

b) 2,0 if the far end of the girder is fixed against rotation (i.e., rigidly attached to a support that is itself relatively rigid).

F.7 Having determined G_U and G_L for a column section, the effective length factor, K, is determined at the intersection of the straight line between the appropriate points on the scales for G_U and G_L with the scale for K.

F.8 The nomograph may be used to determine the effective length factors for the in-plane behaviour of compression members of trusses designed as axially loaded members even though the joints are rigid. In this case, there should be no in-plane eccentricities and all the members of the truss meeting at the joint must not reach their ultimate load simultaneously. If it cannot be shown that all members at the joint do not reach their ultimate load simultaneously, then the effective length factor of the compression members shall be taken as 1,0.

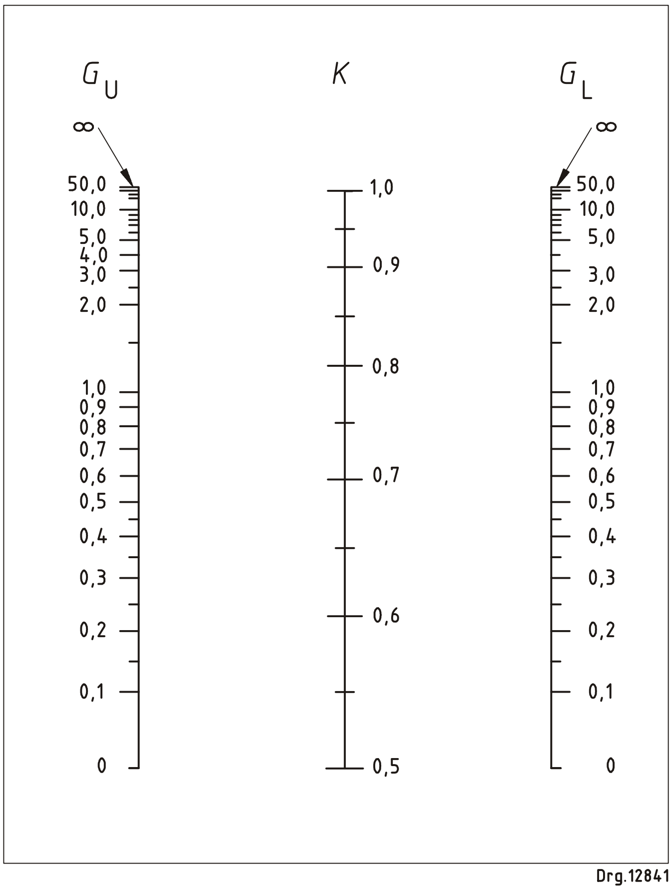


Figure F.1 — Nomograph for effective lengths of columns in continuous frames

Annex G
(informative)

Deflections of composite beams due to shrinkage of concrete

G.1 Shrinkage-induced deflections result from the following process. Concrete decreases in volume as it cures, at first rapidly and then at a decreasing rate. When restrained, tensile strains and therefore tensile stresses are developed in the concrete. (It may even crack if the tensile strength is reached.) A curing slab is restrained by the steel section to which it is connected.

G.2 Figures G.1 and G.2 show the shrinkage strains that develop through the depth for a composite beam and truss respectively and the corresponding equilibrium conditions for unshored construction. It is evident that unshored composite members will deflect downward. (Shoring reduces the shrinkage deflection substantially, especially in the early stages when the rate of shrinkage is the greatest.)

In these figures

ε_f is the free shrinkage strain of the concrete,
ε_s is the tensile strain in the concrete,
ε_r is the resulting restrained shrinkage strain,
$\varepsilon_t, \varepsilon_{tc}$ is the compressive strain at the top of the steel beam or top chord of the truss,
$\varepsilon_b, \varepsilon_{bc}$ is the tensile strain at the bottom of the steel beam or bottom chord of the truss,
T is the tensile force in the concrete,
C, C_{tc} is the compressive force in the beam or in the top chord of the truss,
T_{bc} is the tensile force in the bottom chord of the truss, and
M is the moment in the steel beam.

G.3 Based on figures G.1 and G.2, Kennedy and Brattland proposed an equilibrium method to determine shrinkage deflections. It is iterative because the concrete response is non-linear. Branson's method, also based on equilibrium and strain compatibility, is equivalent when the same values are used for the free shrinkage strain and the modulus of elasticity of the concrete. It is easier to use than the equilibrium method; however, the tensile stress-strain relationship of the concrete is not necessarily satisfied. In spite of this, it gives reasonable results when appropriate values are assumed for the free shrinkage strain and modular ratio.

G.4 The shrinkage deflection is directly proportional to the assumed free shrinkage strain. The free shrinkage strain depends on the concrete properties such as water/cement ratio, percent fines, entrained air, cement content and the curing conditions. A value of 800$\mu\varepsilon$ may be used (see ACI 209R-92) if no other data are available.

G.5 The shrinkage deflection is not sensitive to the modular ratio because both the effective moment of inertia and the distance, y, vary with it. Shaker and Kennedy show that the effective modulus of elasticity, E_{ct}, decreases with increased tensile strain, t, due to increased creep of the concrete. An approximate relationship, for 30 MPa to 40 MPa concrete, from Shaker and Kennedy, is:

$$E_{ct} = 8300 - 4800\ \sigma_{ct};\ 0,3 \leq \sigma_{ct} \leq 1,2$$

which could be used with figures G.1 and G.2 to compute the shrinkage deflection (see Kennedy and Brattland).

(At the maximum tensile stress of 1,2 MPa, reached without cracking, the effective modulus is only about 2 500 MPa or about 10 % of the 28-day modulus in compression and results in a modular ratio of 80)

Changing the modular ratio from 20 to 80, for beams of usual proportions, decreases the shrinkage deflection in the order of 30 % for a given free shrinkage strain. Modular ratios of 40 to 60 (see Ferguson) are considered appropriate and over this range the decrease in the shrinkage deflection is only about 15 %.

Because of interfacial slip and the partial loss of strain compatibility, shrinkage deflections calculated using moduli based on Shaker and Kennedy should be considered an upper bound. Shrinkage deflections calculated using a modular ratio, n_t, of 60 are reasonable.

G.6 Montgomery et al. give an example where the shrinkage deflections were excessive. Jent provides information on shrinkage effects on continuous composite beams.

G.7 References

ACI 209R-92 American Concrete Institute (ACI), 1992, Designing for the effects of creep, shrinkage, and temperature in concrete structures, American Concrete Institute, Detroit, Michigan.

Branson, D.E. 1964, Time-dependent effects on composite concrete beams, Proceedings, Journal of the American Concrete Institute, 61:212-229.

Ferguson, P.M. 1958, Discussion of Miller, L.A., Warping of reinforced concrete due to shrinkage, American Concrete Institute Journal, Vol 30, No. 6, Part 2, 939-950.

Jent, K.A. 1999, Effects of shrinkage, creep and applied loads on continuous deck slab composite beams. M.Sc. thesis, Queens university, Kingston, Canada.

Kennedy, D.J.L. and Brattland, A. 1992, Shrinkage tests of two full-scale composite trusses, Canadian Journal of Civil Engineering, (19)2:296-309.

Montgomery, C.J., Kulak, G.L. 1983, and Shwartsburd, G., Deflection of a composite floor system, Canadian Journal of Civil Engineering, (10)2:192-204.

Shaker, A.F. and Kennedy, D.J.L. 1991, The effective modulus of elasticity of concrete in tension, Structural Engineering Report 172, Department of Civil Engineering, The University of Alberta, Edmonton, Alberta.

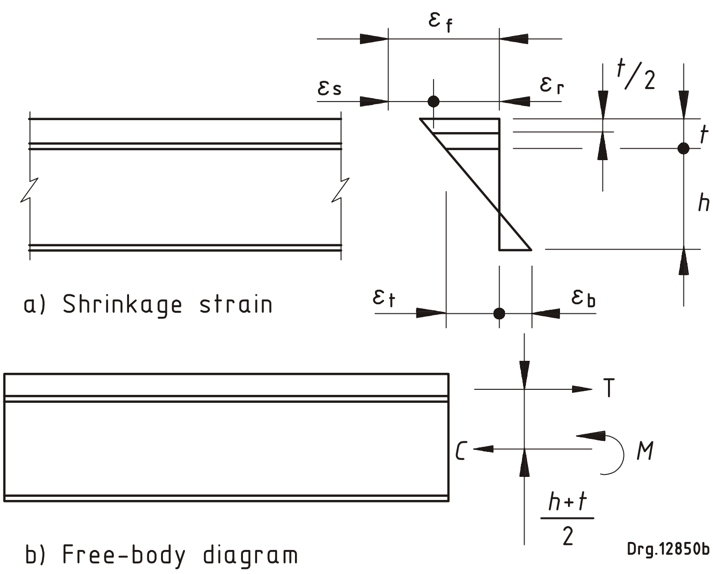


Figure G.1 — Composite beam subject to shrinkage forces (see clauses G.2, G.3 and G.5)

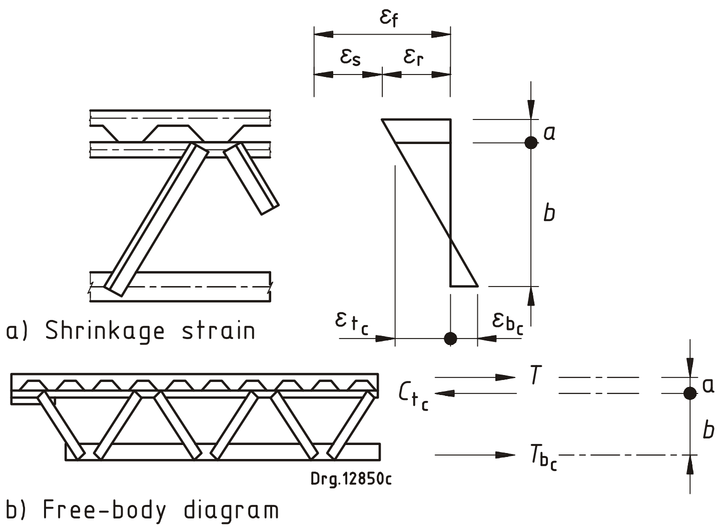


Figure G.2 — Composite truss subject to shrinkage forces (see clauses G.2, G.3 and G.5)

Annex H
(informative)

Crane-supporting structures

H.1 Steel structures that support certain classifications of overhead cranes and hoists require special considerations in order to provide safe and serviceable structures. Electrically-operated top running overhead travelling cranes, underslung cranes and monorails can impose repetitive loads, that may lead to the development and propagation of fatigue cracks in the crane supporting structure. These loads shall be accounted for in the design and construction of the crane support structure. Conditions that apply to these steel structures, where any component is subjected to fatigue loads as defined in clause 26 are given here.

Light duty crane-supporting structures where no element is subjected to fatigue loading are adequately covered by the general provisions of this part of SANS 10162, and special provisions may be considered optional.

The requirements of this part of SANS 10162 for design for fatigue shall apply to all components of the structure that, as shown by the structural analysis, are subject to fatigue. The structural design shall consider, but may not be limited to, appropriate methods of analysis, rotational restraints at crane runway beam supports, crane load eccentricities, distortion leading to fatigue cracking, welded details, built up column section details, bracing systems, deflections and details related to crane rails. The construction standards shall include, but may not be limited to, requirements for materials, detailing, fabrication and erection, bearing and contact surfaces, dimensional tolerances, crane rail installation, and shop and field inspection.

The designer shall obtain information on the class of crane service, frequency of loading and the expected life of the structure in addition to the other crane details that are necessary to design the structure. This information shall be included in the structural design documents.

The class of service for the structure may be described by an equivalent number of crane passages at full load or by the appropriate number of loading cycles at each level of load estimated by analyzing the duty cycles.

H.2 Reference

CISC 2000. *Crane-supporting steel structures*. Canadian Institute of Steel Construction. Willowdale, Ont.

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Frank and Fisher, *Journal of the Structural Division*, ASCE, Vol. 105, No. ST9, September 1979.

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