Annex 1: Symbols and units

A1.1 Symbols The terms and words used in this Code carry the meaning normally assigned within the area of structural steel, and are generally defined the first time they appear within the document.

A1.1.1 Roman Capitals

Term	Definition	Section
A	accidental action value. Width between corners in cold-formed sections. Percentage of steel products holding an officially recognised quality mark.	9.2, 22.3.2, 24.3.1, 34.2, 34.3, 34.5, 34.7.2.1, 34.7.2.2, 35.1.1, 35.1.2, 35.1.4, 35.3, 46.3, 56.1, 60.2.1, 60.3, 61.1, 80.2, A11.4.3.1, A11.4.3.4.
A_0	chord section. Maximum distribution area in compression on a base plate;	64.2, 65.2.2, A-9-9, A-9-17, A-9-19
A'_0	restricted area on which the force of the foundation base plate is applied;	65.2.2
A_1	member 1 cross sectional area;	A-9-2, A-9-3
A_c	gross area for stiffened plate elements subject to uniform compression. Gross section of the compression zone of the stiffened plate, excluding parts of the subpanels supported by an adjacent plate panel;	A6.4.1, A6.4.2
$A_{c,ef}$	area of the effective section of the compressed slender flange, with or without stiffeners, related to local buckling. Area of effective section of the compression zone of the stiffened panel;	21.5, 34.1.2.5, A6.4.1, A6.4.4
$A^*_{c,ef}$	area of the effective section of the compression zone of the stiffened panel for cases in which the influence of shear lag is significant;	A6.4.1
$A_{c,ef,loc}$	sum of effective areas of all stiffeners and subpanels located entirely or partly in the compression area. Effective area of the stiffened plate, excluding parts of the subpanels supported by an adjacent plate panel, calculated taking into account any local buckling of the various subpanels and/or plate stiffening elements;	A6.4.1, A6.4.2

A_{cor}	chord cross-sectional area;	71.2.3, 71.2.3.1, 71.2.3.2
A_d	diagonal cross-sectional area. Diagonal area of the lattice;	18.2.5, 62.1.4, 71.2.3.1, A3.3.1
A_e	equivalent shear cross-sectional area;	67.2.1.2
$A_{E,k}$	characteristic seismic action value;	13.2
A_{ef}	effective area or effective cross sectional area, as applicable. Area of effective reduced section obtained under the application of an axial compression force;	21.5, 22.3.5, 34.1.2.5, 34.3, 34.7.2.3, 35.1.1, 35.1.2, 35.1.4, 35.3, 35.7.1, 35.7.2, 60.3, 73.9.3, 73.11.3, 73.11.4, A6.2, A6.5
$A_{ef,f}$	effective area of the compression flange;	35.2.3
$A_{ef,w,c}$	effective area of the compression part of the web;	35.2.3
A_f	gross area of the tension flange. Area of one flange. Floor area of the fire protection area;	34.4, 34.5, 35.5.2.2, 61.1, A8.4.2, A8.5
$A_{f,net}$	net area of the tension flange;	34.4
A_{fb}	area of dual stiffeners in a tension zone or compresion zone of a support;	62.1.3
$A_{fc,ef}$	effective area of the compression flange;	35.8
A_{fi}	surface area of the fire protection area where fire load is uniformly distributed;	A8.4.6
A_g	gross area of steel profile section or light steel structure;	73.4, 73.11.4
A_h	surface area of horizontal ceiling openings;	A8.5
A_i	chord cross-sectional area; (i=1.2…). Area of the n^{th} portion of a partition of the section such that the interior temperature of each one may be associated with a uniform value (θ_i), at each stage of the process. Area of trapezium forming the cross section of the box beam. Parameter;	18.2.5, 35.3, A3.3.1, A11.4.1
A_j	surface of wall member j, excluding openings;	A8.6.1

A_k	characteristic accidental action value.	13.2
A_L	exposed perimeter area of a steel member by longitudinal unit;	48.1
A_m	cross-sectional area of an upright. Exposed perimeter area of a steel member by longitudinal unit;	18.2.5, 48.1
A_{net}	net cross section:	34.2, 58.2, 58.5.2, 73.13.4
A_{nt}	net area of the zone subject to tension in terms of tear resistance;	58.5.1
A_{nv}	net area of the shear zone in terms of tear resistance;	58.5.1
A_p	internal surface area of the coating per unit of length of the member. Gross section of the plate;	48.2, A6.4.2.1
A_{pl}	base plate surface;	65.2.3
A_r	portion of web of the support between the two moment stiffeners. Excessive thickness of bridge deck plate welding.	62.1.3
A_s	cross-section of a stiffener in terms of calculating buckle resistance. Tension resistant area of the bolt;	35.9.1, 58.6, 58.7, 58.8, 61.2, 62.3, 65.2.1, 65.2.5, 76.7
A_{sl}	area of longitudinal stiffeners situated within a width (b_0) of the flange. Sum of the gross section of all longitudinal stiffeners individually.	21.4, A6.4.2.1
$A_{sl,1}$	gross section of equivalent column on the elastic foundation for calculating critical local buckling tension on plates. Gross area of stiffener cross section and of the adjacent parts of the plate;	A6.4.2.2, A6.4.3
$A_{sl,ef}$	sum of effective areas of all longitudinal stiffeners with a gross area A_{sl} situated within the compresion zone. Area of effective section due to plate local buckling;	A6.4.1, A6.4.3
A_t	total boundary area of fire protection area (walls, ceiling and floor, including openings);	A8.4.2
A_v	shear area. Area of upright cross section. Surface of openings. Surface of vertical front openings.	34.5, 46.4, 71.2.3.1, A8.4.6, A8.5, A8.6.1, A-9-9
A_{vc}	shear area of the column;	62.1.2, 62.1.4, 62.3

A_w	web area. Button weld hole area;	34.5, 34.7.1, 35.8, 56.1, 59.10
B	deck width. Width between the corner and the free edge of cold-formed sections;	38.4, 80.2
B_D	distortional bi-moment to be resisted by the beam along longitudinal axis z;	A3.3.1
B_{Ed}	warping torsion bi-moment;	34.6
$B_{p,Rd}$	punching shear resistance of the part beneath the nut or the bolt head;	58.7
C_1	correction factor for the law of moments;	35.3
C_d	admissible limit value for the limit state to be checked (strains, vibrations, etc.);	8.1.3,
C.E.	execution class;	91.2.2.5
C_E	set square coefficient;	61.6
CEV	equivalent carbon value;	26.5.5, 27.1, 27.2.1, 27.2.3, 27.2.2, 27.3 , 59.3.2
$C_{f,d}$	factor of friction between the base plate and concrete;	65.2.1
C_{LT}	factor;	35.3
$C_{mi,0}$	factor;	35.3
C_{mLT}	equivalent uniform moment factor;	35.3
C_{my}	equivalent uniform moment factor;	35.3
C_{mz}	equivalent uniform moment factor;	35.3
C_{yy}	factor;	35.3
C_{yz}	factor;	35.3
C_{zy}	factor;	35.3
C_{zz}	factor;	35.3

D	concave or convex deviation of flatness on a cold-formed profile. Fire diameter;	80.2, A8.6.2
D_1	tool length;	92.2
D_2	distance from the tool to the wall;	92.2
D_d	damage accumulated for detail as a result of load cycles;	42.3, 42.6
D_r	existing slope between orthotropic bridge decks, after assembly;	80.4
E	longitudinal elastic modulus of steel;	18.2.4, 18.2.5, 20.3, 26.2, 26.3, 26.5.2, 32.4, 35.3, 35.5.2.1, 35.6, 35.8, 57.4, 61.6, 62.1.2, 62.3, 65.2.5, 66.2, 71.2.3, 71.2.3.1, 71.2.3.2, 73.7, 73.9.2, 73.10, 73.11.3, A6.4.2.1, A6.4.3, A7.2.1, A3.3.1, A5.2, A-9-8
E_a	modulus of elasticity of steel at 20ºC;	45.1
$E_{a,\theta}$	elastic modulus in the linear phase of the stress-strain diagram for the temperature θ_a;	45.1
E_c	elastic modulus between the concrete and the compressed base plate;	65.2.5
E_d	design value of the effect of actions. Elastic modulus of steel;	8.1.2, 8.1.3,
$E_{d,desestab}$	design value of the effect of destabilising actions;	8.1.2, Art. 33
$E_{d,estab}$	design value of the effect of stabilising actions;	8.1.2, Art. 33
$E_{fi,d}$	Effects of the action of fire defined in 43.2 together with the concomitant mechanical actions specified in Art. 44, with the partial safety factors indicated therein;	Art. 46, 46.8.1, Art. 47, A8.5
E_g	internal energy of gas;	A8.7.1
$EI\eta''_{cr}$	moment due to the strain η_{cr} in the critical cross section;	22.3.5

E_s	longitudinal elastic modulus;	32.2
F	Strength. Factor of proportionality;	59.8.2, A3.3.3
$F_{b,Ed,ser}$	flattening force on a removable pin at serviceability limit state;	58.9
$F_{b,Rd}$	flattening force of the piece in the area adjacent to the screw;	58.6, 58.9, 58.10
$F_{b,Rd,ser}$	flattening resistance of removable pin at serviceability limit state;	58.9
$F_{c,Ed}$	force transmitted to the bracket;	60.3
$F_{c,fb,Rd}$	design compression strength of the flange and web of the beam;	61.2.1, 62.2.2
$F_{C,Rd}$	strength of the compresion zone resulting from stresses transmitted from compressed elements of the support to the foundation through the base plate;	65.2.2
$F_{c,wc,Rd}$	maximum compression stress that can be resisted by the compresion zone on the support;	62.1.2, 62.2.2
$F_{ch,Rd}$	tensile strength of steel-bolt set in joints with an end sheet and high-strength pre-loaded bolts;	58.7
F_{cr}	Critical load of elastic instability for a certain form of lateral instability in a non-translational structure, under the configuration of the combination of actions to be considered. Critical transverse load of local web local buckling against the concentrated load;	23.2, 35.6
F_d	design value of an action F.	Section 12
$F_{e,Rd}$	failure resistance on the edge of a weld point in a resistance weld;	73.13.4
F_{Ed}	design value of applied transverse strength. Design load acting on the structure for the combination of actions. Design compression reaction value of the support;	21.6, 23.2, 35.6, 35.7.2, 58.3, 58.9, 58.10, 59.3.4, 60.2.2, 61.3, 62.1.3, 66.2
$F_{Ed,ser}$	design compression reaction value of the support at serviceability limit state;	66.2
$F_{H,Ed}$	design horizontal strength, estimated on the lower level of each storey, resulting from the horizontal loads acting on each level, including the effects of imperfections;	23.2.1
$F_{i,Ed}$	stress on each bolt resulting from the relative rotation between pieces to be joined in a bolted eccentric butt joint;	60.2.2, 60.2.2

F_k	characteristic value of an action. Point force;	Section 11
$F_{net,Rd}$	net sectional strength of a weld point in a resistance weld;	73.13.4
F_{Rd}	strength of the web against concentrated transverse loads;	35.6, 58.10, 60.1.2, 60.2.2, 62.1.3, 62.1.5, 62.2.1, 62.4.3
F_{Rdu}	maximum concentrated compression strength that can act on the concrete according to Spanish EHE-08 standards;	65.2.2
F_s	point load;	35.6
$F_{s,Ed}$	stress to be transmitted by the bolt;	58.2
$F_{s,Rd}$	slip resistance of a high-strength pre-loaded bolt;	Section 39, 58.2, 58.8
$F_{s,Sd}$	design force in service of the bolt;	Section 39
$F_{t,Ed}$	tensile force on a bolt in the direction of its axis;	58.7, 58.7.1, 58.8
$F_{T,ep,Rd}$	design tension resistance for a single row of bolts of a bending end sheet;	61.2.1, 62.2.2
$F_{t,fc,Rd}$	strength of the support flange;	62.1.2, 62.2.2
$F_{T,i,Rd}$	design tensile resistance for an equivalent T-shaped bearing;	61.2
$F_{t,Rd}$	design strength per bolt;	58.7, 58.7.1, 61.2
$F_{T,Rd}$	design tension resistance of an equivalent T-shaped bearing;	61.2, 61.2.1, 62.2.1, 62.2.2
$F_{t,wb,Rd}$	design tension resistance for a single row of bolts of a tensile beam;	61.2.1, 62.2.2
$F_{t,wc,Rd}$	chord support;	62.1.2, 62.2.2
$F_{tb,Rd}$	flattening and tearing strength of a weld point in a resistance weld;	73.13.4
$F_{tr,Rd}$	effective design tensile resistance of a row of bolts r;	61.2.1, 62.2.2
$F_{v,Ed}$	calculated vertical resistance, estimated on the lower level of each floor, resulting from the vertical loads acting on each level. Design stress on a bolt in the normal direction of its axis;	23.2.1, 58.6, 58.7.1, 58.9

$F_{v,Rd}$	shear force of the bolt or weld point in a resistance weld;	58.6, 58.7, 58.9, 58.10, 73.13.4
$F_{V2,Rd}$	shear and flattening force of the bolt against the base plate or against the lock washer;	65.2.1
$F_{w,Ed}$	design force to be transmitted in a button or slot weld. Design shear on the chord or weld point in overlapping joints made with arc welding;	59.10, 60.1, 60.1.1, 60.2.1, 73.13.5.1, 73.13.5.2
$F_{w,Rd}$	shear force of the chord or weld point in overlapping joints made with arc welding;	73.13.5.1, 73.13.5.2
$F_{w,Sd}$	strength of a button or slot weld;	59.10
G	permanent action value. Modulus of transverse elasticity of steel. Gross sectional centre of gravity. Modulus of transverse elasticity of neoprene;	9.2, 18.2.4, 18.2.5, 20.7, 32.4, 66.1, 73.11.3, A3.3.3
G_*	value of the non-constant permanent action;	9.2,
G´	centroid of the effective section;	20.7
G_{kj}	characteristic permanent action value.	13.2
$G^{*}_{j,k}$	characteristic value of non-constant permanent actions;	13.2
H	altitude (metres above sea level). Height of the building. Horizontal force. Difference between axial forces on each side of the node. Horizontal force that the support is able of transmitting. Height of the fire protection area. Distance between fire source and ceiling;	Art. 11, 37.1, 37.2.2, 50.2, 60.3, 66.1, A8.5, A8.6.2
H_{Ed}	design value resulting from the total horizontal actions, at the base of the building, for the combination of actions considered;	22.3.1
H_i	height of one storey of the building;	37.1, 37.2.2
H_{td}	design value of transverse strength equivalent to the initial defects of verticality in compressed elements;	22.3.3
H_u	net calorific value of the material;	A8.4.4, A8.4.6
H_{u0}	net calorific value of the dry material;	A8.4.4
H_{ui}	net calorific value;	A8.4.2
HV	Vickers hardness test;	66.2, 77.5.5

I	second moment of area of the cross section;	34.5, 35.3, 61.1, 73.11.3, A5.2
ICES-EA	sustainability contribution index of steel structure;	A11.1, A11.3, A11.5, A11.6.1, A11.6.2
ISMA-EA	environmental sensitivity index of steel structure;	A11.1, A11.3, A11.4, A11.4.1, A11.5
I_b	second moment of area of the beam connected at the joint. Inertia to distortional warping of the section in a symmetric single-cell box beam;	57.4, A3.3.1
I_c	second moment of area of the column connected at the joint;	57.4, 65.2.5
I_{cor}	inertia of the chord on the plane;	71.2.3.2
I_{ef}	second moment of area of the effective section. Effective inertia of the compound member;	20.7, 71.2.3, 71.2.3.1, 71.2.3.2, A6.2
$I_{ef,f}$	second moment of area of the compressed flange reduced around the weak axis of the section;	35.2.3
I_f	second moment of area of the set of both flanges in relation to the axis of inertia of the piece;	61.1
I_i	second moment of area of the cross section of a chord (i=1,2);	18.2.5
I_m	second moment of area of the cross section of an upright;	18.2.5
I_{mvn}	minimum inertia;	67.2.1.2
I_{net}	second moment of area of the net section of the transverse stiffener;	A7.2.4
I_p	polar second moment of area of the chord area in relation to its centroid. Inertia of the fastener on the plane. Bending inertia of the plate. Polar second moment of area of the stiffener alone around the edge fixed to the stiffened plate;	60.2.1, 71.2.3.2, A6.4.2.1, A7.2.1
I_{sl}	second moment of area of longitudinal stiffening around the z-z axis. Second moment of area of the plate;	35.5.2.1, 35.9.3.3, A6.4.2.1
$I_{sl,1}$	second moment of area of the gross section of the column around an axis passing through its centroid and parallel to the plane of the sheet. Inertia of the gross cross section of the longitudinal stiffener and adjacent parts of the sheet, with respect to the bending axis, resulting in a warping of the stiffener outside the plane of the sheet;	A6.4.2.2, A6.4.3

I_{st}	second moment of area of the transverse stiffener;	A7.2.1
I_T	St. Venant torsional constant for the stiffener alone;	A7.2.1
I_t	St. Venant's torsional inertia;	18.2.4, 73.11.3
I_w	warping inertia. Second moment of area of the framework member for the web of the box;	18.2.4, 34.6, 61.1
I_y	effective load length;	35.6, 35.7.2
K	Stiffness effective coefficient of the beam;	A5.2
K_i	parameter;	A11.4.1
K_1, K_2	Stiffness coefficients for the longitudinal sections adjacent to the support;	A5.2
K_b	average I_b/L_b value for all beams at the top of the storey;	57.4
K_c	average I_c/L_c value of columns at the top of the storey. Coefficient of stiffness of column I/L;	57.4, A5.2
K_D	stiffness constant of the diaphragm;	A3.3.1
$K_{i,j}$	Stiffness effective coefficient of the beam (with i=1.2 and j=1.2);	A5.2
K_v	notch impact value;	26.2
L	longitudinal span length. Bracket length. Element length. Weld chord lengths. Total slot length. Base plate depth. Gauge length;	21.1, 22.3.2, 22.4, 27.2.2, 27.2.4, 34.1.2.5, 35.3, 37.2.1, 37.2.2, 37.3.1, 37.3.2, 40.2, 42.6, 45.1, 46.3, 59.3.6, 60.3, 61.6, 65.2.3, Art. 67, 71.1, 71.2.3, 71.2.3.2, 72.3, 73.8, 73.11.3, 80.2, 80.4, A5.2
L1	free distance between partial chord ends in discontinuous welds, on the same face or on different faces, in tension pieces. Distance between adjacent stiffeners;	59.3.4, 80.2
L_2	free distance between partial chord ends in discontinuous welds, on the same face or on different faces, in compresion or shear pieces.	59.3.4, 80.2

	Distance between adjacent stiffeners;	
L_b	length of the beam connected at the joint. Bolt extension length. Bolt tightening length, distance from the mid-point of the nut to the mid-point of the head. Rod length. Lattice diagonal length;	57.4, 62.3, 65.2.5, A3.3.1
L_c	length of the column connected at the joint;	57.4, 65.2.5
L_{cor}	chord buckling length.	71.2.3.1
L_{cr}	buckling length at the plane of the bending buckle;	35.1.3, A5.1, A5.2
L_f	length of flames in a localised fire;	A8.6.2
L_{fi}	buckling length in a fire situation;	46.3
L_h	horizontal flame length;	A8.6.2
L_j	joint length, measured in the direction of force to be transmitted between centres of end bolts;	60.1.2
L_p	geometric diagonal length of the cross section of the box;	A3.3.1
L_w	length of each partial chord in discontinuous welds;	59.3.4, 59.8.1, 59.8.2, 59.10, 60.3, 61.4, 73.13.5.2
$L_{w,e}$	length of end edge angle weld, parallel to the direction of stress;	73.13.5.1
$L_{w,s}$	length of side angle weld, parallel to the direction of stress;	73.13.5.1
L_{we}	partial chord length in discontinuous welds on the edges of the pieces to be joined;	59.3.4
L_{wef}	effective length of an angled weld chord;	59.8.1
L_{wi}	length of a weld chord at the joint;	60.1.1
M	Bending moment;	20.7, 57.2, 60.3
$M_{0,Ed}$	design bending moment in the chord;	64.2
$M_{b,fi,t,Rd}$	design bending strength against the lateral buckling of a member with a grade 1, 2, or 3 section, with a maximum steel temperature in the section's	46.5

	compression flange ($\theta_{a,com}$) at a time (t) in the fire process;	
$M_{b,Rd}$	design bending strength against lateral buckling;	35.2.1, 73.11.3
$M_{b,V,Rd}$	design lateral buckling strength of a plate without lateral bracing subject to bending stress around the strong axis at room temperature, including the reduction resulting from shear stress, where applicable;	46.5
$M_{b1,Ed}$	design bending moment on one side of the support;	62.1.2, 62.1.4, 62.3
$M_{b2,Ed}$	design bending moment on the other side of the support;	62.1.2, 62.1.4, 62.3
$M_{c,Rd}$	design bending strength of the cross-section;	34.4, 34.7.2.1, 35.2.3, 62.1.2, 73.11.2
$M_{c,T,Rd}$	design bending strength of a cross-section subject to bending and warping;	34.6
M_{cr}	critical elastic lateral buckling moment;	35.2.2. 35.2.2.1
M_{Ed}	design value of the bending moment. Design value of the maximum bending moment of the compound member, considering second-order effects;	34.4, 34.7.2.1, 34.7.2.2, 34.7.2.3, 35.2.1, 35.2.2.1, 35.2.2.2, 35.7.1, 35.7.2, 53.2, 56.1, 57.3, 57.5, 58.9, 60.2.1, 60.2.2, 61.1, 61.2.1, 62.1.1, 62.2.2, 65.2.1, 71.2.3, 73.11.3, A6.5, A-9-13
M'_{Ed}	design value of the maximum bending moment at the centre of the compound member, without considering second-order effects;	71.2.3
$M_{Ed,1}$	design bending moment in member 1;	A-9-2, A-9-3
$M_{Ed,ser}$	bending moment affecting removable pin at serviceability limit state;	58.9
M_{Edw}	design bending moment resulting from the bending force in the splice section and from shear eccentricity in a butt joint with splice plate;	61.1
M_{el}	elastic bending moment of the section of the piece;	56.1
$M_{f,Rd}$	design bending strength of the cross section considering effective flange sections only;	35.5.2.2, 35.7.1

$M_{f,Rk}$	characteristic bending strength of the cross section considering effective flange sections only;	35.5.2.2
$M_{fi,Ed}$	design bending moment in a fire situation;	46.4, 46.6
$M_{fi,t,Rd}$	design value of the bending moment strength for a time (t) of the fire process;	46.4
$M_{fi,\theta,Rd}$	design bending strength of a grade 1, 2 or 3 section with lateral buckling and with a uniform temperature ($\theta_{a,t}$);	46.4
M_i	section bending moment for individual load "i" for calculating coefficient Ψ_{el}	21.3.3
$M_{i,Ed}$	design bending moment in each joint in the bolted eccentric butt joint;	60.2.2
$M_{ip,1,Rd}$	design bending strength of the joint, expressed in terms of the interior bending moment of element 1 at the plane of the joint;	A-9-3, A-9-4, A-9-10, A-9-11, A-9-18
$M_{ip,i,Ed}$	design value of bending moment at the plane of the joint;	64.6.2, 64.7.2.1, 64.8
$M_{ip,i,Rd}$	bending moment joint strength at the plane of the joint;	64.6.2, 64.7.2.1, 64.8
$M_{j,Ed}$	design bending moment at the joint;	57.5, 62.2.2
$M_{j,Rd}$	design bending moment resistance of a joint;	61.2.1, 62.1.5, 62.2.1, 62.2.2, 62.4.1, 62.4.3
$M_{k,i}$	mass of combustible material;	A8.4.3
$M_{N,Rd}$	plastic resistance of the effective section, due to the existence of axial force N_{Ed};	34.7.2.1, 35.7.1
$M_{op,1,Rd}$	design bending strength of the joint, expressed in terms of the interior bending moment of member 1 outside the plane of the joint;	A-9-2, A-9-3, A-9-4, A-9-11
$M_{op,i,Ed}$	design value of bending moment at the normal plane of the joint;	64.6.2, 64.7.2.1
$M_{op,i,Rd}$	bending moment joint strength at the normal plane of the joint;	64.6.2, 64.7.2.1
M_{pl}	plastic moment of the section of the piece;	56.1
$M_{pl,Rd}$	design plastic bend strength;	35.3, 35.7.1, 53.2, 57.3, 61.2, 61.6, 62.4.1, 73.11.2
M_{Rd}	ultimate moment in the moment-rotation diagram for the joint subject to bending moment;	57.2, 57.3, 57.5, 58.9

$M_{Rd,ser}$	bending resistance of removable pin at serviceability limit state;	58.9
M_{Rk}	characteristic critical cross section bending strength;	22.3.5, 35.3
M_t	torsional moment applied to the bolt;	76.7.1
$M_{V,Rd}$	ultimate bending moment of the single bending section at room temperature, including the reduction resulting from shear stress, where applicable;	46.4
$M_{x,Ed}$	design torsional moment concomitant with $N_{c,Ed}$;	65.2.1
$M_{y,Ed}$	maximum design bending moment strength between bracing points. Maximum design bending moment strength along the member around the y-y axis;	35.2.3, 35.3
$M_{y,V,Rd}$	design plastic bending strength for H sections with equal flanges bending around the main axis of inertia of the section, considering the interaction with shear stress and torsional stress;	34.7.1
$M_{z,Ed}$	maximum design bending moment along the member around the z-z axis;	35.3
N	normal stress. Number of specifications;	20.7, 41.1, 48.4, 59.3.5, 60.3, 65.2.2, 72.4.3, A5.2, A6.6
N_0	design pre-loaded force on the bolt;	58.2, 58.8, 65.2.3, 76.7, 76.7.1
$N_{0,Ed}$	axial design force on the chord;	64.2, A-9-5, A-9-6, A-9-16
$N_{0,Rd}$	design axial force strength on the chord;	A-9-9, A-9-17, A-9-19
N_1	axial compression force applied to a section of the chord;	72.4.1
N_1	axial force applied to member 1;	A-9-1, A-9-3, A-9-5, A-9-6, A-9-7, A-9-10, A-9-12, A-9-14, A-9-15, A-9-19
$N_{1,Ed}$	design axial force on element 1;	A-9-5, A-9-6, A-9-12, A-9-16
$N_{1,Rd}$	design axial force strength on element 1;	A-9-1, A-9-2, A-9-3, A-9-5, A-9-7, A-9-10, A-9-12, A-9-17
N_2	axial compression force applied to the other section of the beam. Axial force applied to element 2;	72.4.3, A-9-1, A-9-5, A-9-6, A-9-7, A-

		9-12, A-9-15, A-9-16, A-9-17
$N_{2,Ed}$	design axial force on element 2;	A-9-5, A-9-6, A-9-12, A-9-16
$N_{2,Rd}$	axial force design strength on element 2;	A-9-1, A-9-5, A-9-12
N_3	axial force applied to element 3;	A-9-5, A-9-12
$N_{3,Ed}$	design axial force on element 3;	A-9-9
$N_{b,fi,t,Rd}$	design buckling strength of a element subject to compression whose section, of an area A, is grade 1, 2, or 3.	46.3
$N_{b,Rd}$	design buckling strength of the compresion element;	35.1.1, 71.2.3, 71.2.3.1, 73.11.3
$N_{c,Ed}$	absolute value of the design compression stress transmitted by the base plate to the foundation;	65.2.1
$N_{c,Rd}$	design strength of compresion section;	34.2
$N_{c,Sd}$	absolute value of the design compression stress transmitted by the base plate to the foundation, including any pre-stressed force from the anchor bolts;	65.2.1
$N_{cor,Ed}$	design value of the axial compression force on a chord between two consecutive links;	71.2.3, 71.2.3.2
N_{cr}	critical elastic axial force for the form of buckling in question. Effective elastic axial force of the compound member;	22.3.2, 24.3.1, 35.1.2, 35.1.3, 35.1.4, 35.3, 41.1, 71.2.3, 73.11.3, A5.2
$N_{cr,F}$	critical elastic axial buckling stress due to bending;	73.11.3
$N_{cr,T}$	critical elastic axial buckling stress due to torsion;	35.1.4, 35.3, 73.11.3
$N_{cr,TF}$	critical elastic axial buckling stress due to torsion and bending;	35.1.4, 73.11.3
N_{Ed}	design value of the axial compression force on the element in question. Design value of the axial compression force on the centre of the compound member;	22.3.1, 22.3.2, 22.3.3, 22.4.1, 24.3.1, 34.1.2.4, 34.2, 34.3, 34.7.2.1, 34.7.2.2, 34.7.2.3, 35.1.1, 35.1.2, 35.3, 35.5.2.2, 35.7.1, 35.7.2, 53.2, 56.1,

		58.5.1, 61.1, 61.2.1, 62.2.2, 62.3, 62.4.1, 65.2.2, 71.2.3, 73.11.3, A6.5, A7.2.1, A-19-3
N_E	critical elastic axial force (Euler critical buckling load) of the stiffening member;	40.1, A5.2
$N_{Ed,1}$	design value of the axial compression force on element 1;	A-9-2, A-9-3
N_{Edf}	axial force resisted by each of the flange joints in a butt joint with splice plate;	61.1
N_{Edw}	guided axial force following the direction of the piece distributed evenly among all bolts in a joint with splice plate;	61.1
$N_{Ef,Rd}$	tear resistance;	58.5.1
$N_{fi,Ed}$	design value of the axial force in a fire situation;	46.4, 46.6
$N_{fi,t,Rd}$	design strength of a section subject to pure tension and with an uneven temperature distribution at time (t) during the fire process;	46.2
N_i	axial compression force applied to element i of the joint;	A-9-7, A-9-8
$N_{i,Ed}$	design axial force on the brace member i;	64.2, 64.6.2, 64.7.2.1, 64.8, A-9-5, A-9-13
$N_{i,Rd}$	design value of the reistance of axial force joint;	64.6.2, 64.7.2.1, 64.8, A-9-1, A-9-5, A-9-7, A-9-8, A-9-9, A-9-13, A-9-14, A-9-15, A-9-17, A-9-19
N_j	axial compression force applied to element j of the joint;	A-9-7, A-9-17
$N_{j,Ed}$	design value of the axial force at the joint;	61.2.1, 62.2.2
$N_{j,Rd}$	design value of resistant axial force at the joint, assuming no moment is applied;	61.2.1, 62.2.2
N_{max}	maximum vertical reaction on the support in N;	66.1
N_{min}	minimum vertical reaction on the support in N;	66.1

$N_{net,Rd}$	design strength under tension of the net area;	34.2
N_p	plastic axial force of the section of the piece;	56.1
$N_{p,Ed}$	axial force disconnecting the components of the diagonals or uprights parallel to the chord axis;	64.2
$N_{pl,0,Rd}$	design plastic resistance of the section at the chord;	A-9-5, A-9-6, A-9-16
$N_{pl,Rd}$	design plastic resistance of the gross section;	34.2, 34.7.2.1, 46.2, 53.2, 61.2.1, 62.2.2, 62.3, 62.4.1, A-9-13
N_{Rd}	design resistance of the joint;	60.1.2
N_{RK}	characteristic axial force resistance of the critical cross section;	22.3.5, 35.3
N_t	axial tension force on the diagonal of length d_t;	72.4.3
$N_{t,Rd}$	design strength of the tensile section;	34.2
$N_{u,Rd}$	ultimate design strength of the net cross section;	34.2, 58.5.2
$N_{x,Ed}$	design axial force for a joint in x;	A-9-5
$\tilde{N}$	number of error cycles in each stress range;	42.6
$\tilde{N}_\sigma$	number of consecutive times Δ_σ direct stress ranges have to be applied to exhaust the fatigue resistance of the detail following the corresponding S-N curve, reduced using the coefficient γ_{Mf};	42.3
$\tilde{N}_\tau$	number of consecutive times Δ_τ shear stress ranges have to be applied to exhaust the fatigue resistance of the detail following the corresponding S-N curve, reduced using the coefficient γ_{Mf};	42.3
O	factor of openings;	A8.5, A8.6.1
O_{lvm}	limit factor of openings;	A8.6.1
P_i	individual point load. Value assumed by the representative function for each indicator;	70.4, A11.4.1, A11.4.3.1, A11.4.3.2, A11.4.3.3, A11.4.3.4, A11.4.3.5,

		A11.4.3.6
P_k	characteristic maximum humidity percentage of protective materials admitted in designs. Total gravitational load above the ground;	45.2, 48.2, 50.2
P_r	deviation in the adjustment of orthotropic decks in the assembly of bridges;	80.4
Q	variable action value. Leverage. Heat release rate;	9.2, 61.2, A8.4.6, A8.6, A8.6.2, A8.7.1
Q_c	convection part of the heat release rate;	A8.6.2
$Q_{fi,k}$	characteristic fire load;	A8.4.2
$Q_H{}^*$	non-dimensional heat release rate;	A8.6.2
Q_k	characteristic variable action value;	42.3
$Q_{k,1}$	characteristic value of the determinant variable action;	13.2, Art. 44
Q_{max}	maximum heat release rate;	A8.4.6
Q_{rad}	loss of energy through openings;	A8.7.1
Q_{wall}	loss of energy through surrounding surfaces;	A8.7.1
R_d	design structural response;	8.1.2, 15.2
R_D	reactions in springs obtained in the equivalent beam model, for the dimensioning of diaphragms;	A3.3.3
$R_{D,rvgido}$	reactions in springs obtained in the equivalent beam model, suppressing fixed supports in sections where diaphragms are located;	A3.3.3
$R_{fi,d,0}$	$R_{fi,d,t}$ value for t=0, i.e. at room temperature;	46.8.1, Art. 47
$R_{fi,d,t}$	respective resistances, assuming the member subject to temperature distribution (θ) at the specific time (t) of the fire process;	Art. 46, 46.8.1, A8.5
RHR_f	Maximum rate of generation of heat produced in 1 m_2 of fire, in the case of fuel-controlled fire;	A8.4.6
R_k	characteristic structural response value;	15.2

S	elastic area moment of the cross section above the point in question. Stress range. Section factor expressed in m_{-1}. Required shear stiffness of the strap;	34.5, 42.5, 48.1, 48.4, 56.1, 73.11.3
S_{ch}	stiffness provided by the sheet;	73.11.3
S_D	effective stresses on the different elements that make up the diaphragm;	A3.3.3
$S_{D,rvgido}$	stresses obtained in the case of rigid diaphragms;	A3.3.3
S_j	rotational stiffness of the joint;	57.2, 57.5, 65.2.5
$S_{j,ini}$	initial rotational stiffness of the joint;	57.2, 57.4, 57.5, 62.3, 65.2.5
S_p	section factor of the protected member in m^{-1};	48.2, 48.4
S_v	shear stiffness of triangulation used for the link or battened panel	71.2.3, 71.2.3.1, 71.2.3.2
T	thickness of orthotropic bridge decks. Temperature;	80.4, A8.7.1
$T_{c,Rd}$	design strength tension of cross-section;	34.6
T_{Ed}	design value of torsional stress. Design value of tensile force in the bolt;	34.6, 65.2.3
T_o	minimum service temperature to consider in the site of installation of the structure;	32.3
T_{ref}	reference temperature in the steel;	32.3
$T_{t,Ed}$	component of torsional force corresponding to St. Venant's uniform torsion;	34.6
$T_{w,Ed}$	component of torsional force corresponding to warping torsion;	34.6
V	steel volume of the element per unit of length in m^3/m;	48.1, 48.2
$V_{0,Ed}$	design value of the shear stress in the chord;	A-9-5, A-9-6, A-9-16
$V_{b,Rd}$	design web local buckling strength;	35.5.2, 73.10
$V_{bf,Rd}$	contribution of flanges to shear local buckling strength of the element;	35.5.2, 35.5.2.2
$V_{bw,Rd}$	contribution of the web to shear local buckling strength of the member;	35.5.2, 35.7.1

V_c	design wind speed;	38.4
$V_{c,Rd}$	design shear strength of the section;	34.5, 46.4
$V_{c1,Ed}$	shear force applied at the joint by connected members;	62.1.4
$V_{c2,Ed}$	shear force applied at the joint by connected elements;	62.1.4
V_{Ed}	design value of the total vertical actions, at the base of the building, for the combination of actions considered. Design value of the shear force;	22.3.1, 34.5, 34.6, 34.7.1, 34.7.3, 35.5.2, 35.7.1, 35.9.3.3, 35.9.3.5, 53.2, 56.1, 61.1, 61.2.1, 61.3, 61.4, 61.5, 61.6, 62.1.1, 62.2.2, 71.2.3, 71.2.3.2, A7.2.4, A-9-19
$V_{Ed,G}$	cutting forces due to non-seismic actions;	53.2
$V_{fi,Ed}$	design value of the shear force in a fire situation;	46.4
$V_{fi,t,Rd}$	design strength of a grade 1, 2, or 3 shear stressed section at a given time (t) during the fire process;	46.4
V_i	factor;	A11.4.1
V_k	combined shear for the storey;	50.2
V_p	plastic shear of the section of the piece;	56.1
$V_{pl,0,Rd}$	design plastic shear strength at the chord;	A-9-5, A-9-6, A-9-16
$V_{pl,Rd}$	design plastic shear strength;	34.5, 34.7.1, 34.7.3, 53.2, A-9-19
$V_{pl,T,Rd}$	design strength of the section against shear force and torsional moment;	34.6, 34.7.1
V_R	maximum shear force that can be transmitted through friction by the base plate to the support foundation;	65.2.1
$V_{Rd,w}$	weld strength;	61.6
V_{Rd1}	beam web resistance to local flattening;	61.5, 61.6

V_{Rd2}	angular bearing weld chord resistance;	61.5, 61.6
V_{Rd3}	shear strength of the angle bearing flange;	61.5
$V_{wp,Ed}$	design shear strength of the web of a non-rigid column;	62.1.4
$V_{wp,Rd}$	design plastic shear strength of the web of a non-rigid column;	62.1.4, 62.2.2
$V_{y,Ed}$	design shear force component along the y axis;	65.2.1
$V_{z,Ed}$	design shear force component along the z axis;	65.2.1
W	shear modulus. Width of the piece to be joined;	35.2.1, 35.2.3, 35.3, 56.1, 59.3.6
W_{ef}	shear modulus of the effective section;	20.7, 34.7.2.3, 35.3, 73.9.3, 73.11.3, A6.2, A6.4.1, A6.5
$W_{ef,min}$	minimum elastic shear modulus of the effective cross section;	22.3.5, 34.4
W_{el}	elastic shear modulus to bending;	34.7.2.2, 35.3, 58.9, 73.11.2, A5.2
$W_{el,0}$	elastic shear modulus of the chord;	64.2
$W_{el,1}$	elastic shear modulus of the section of member 1;	A-9-2, A-9-3
$W_{el,min}$	minimum elastic shear modulus of the section;	22.3.5, 34.4
W_{pl}	plastic shear modulus of the section;	22.3.5, 34.4, 34.7.1, 35.3, 73.11.2
$W_{pl,1}$	plastic shear modulus of the section of member 1;	A-9-11
Z	contraction expressed as a percentage;	26.2, 26.5.2

A1.1.2 Roman Lower Case

a	value of geometric data. Size. Factor. Distance between transverse stiffeners. Weld neck thickness. Upper size on ground of neoprene support. Minimum distance permitted between sections or between one section and an adjacent surface. Length of the sheet with or without stiffeners. Distance between	16.1, 18.2.5, 34.7.2.1, 35.5.2.1, 35.5.2.2, 35.6, 35.9.3.3, 59.7, 59.8.2, 59.9.2, 59.10, 61.2, 61.3,

	transverse stiffeners; Factor of social contribution of the structure to sustainability;	61.4, 61.5, 61.6, 64.3, 64.8, 66.1, 71.2.3.1, 71.2.3.2, 80.2, 92.2, A6.3, A6.4.2.1, A6.4.2.2, A6.4.3, A6.5, A11.5
a_1, a_2	lengths of the panels adjacent to the transverse stiffener;	A7.2.1
a_1, a_2,…a_5	individual factors of social contribution of the structure to sustainability;	A11.5, A11.6.1
a_b	throat thickness of the joint of beam flanges to the support;	62.1.1, 62.1.2
a_c	throat thickness of chords at flange-web joints in the support. Buckling length of the equivalent column;	62.1.1, 62.1.2, 62.2.1
a_d	design value of geometric data;	16.1
a_f	factor;	34.7.2.1
a_i	throat thickness of one of the weld chords at the joint;	60.1.1
a_k	characteristic geometric data value;	16.1
a_{LT}	factor;	35.3
a_{nom}	nominal geometric data value;	16.1
a_{st}	gross cross sectional area, per unit of length, of stiffeners situated in the area affected by the load beneath the cover plate, for loads applied on the plane of the web of a section;	21.6
a_w	factor;	34.7.2.1
b	size. Section width. Double the height of the compresion panel (or subpanel). Weld chord width. Slot width. Distance between bolts. Lower size of the base plate. Lower size on the ground of the neoprene support. Width of the welded plate or strip. Width of the sheet with or without stiffeners. Factor. Coefficient of contribution of the structure to sustainability through extension of the working life;	18.2.5, 20.7, 21.4, 34.5, 35.1.2, 35.2.2, 35.2.2.1, 40.2, 42.6, 48.1, 48.2, 58.9, 59.3.4, 59.5, 60.3, 61.3, 61.5, 65.2.1, 66.1, 73.5, 73.6, 73.13.5.1, 80.2, 80.3, A6.3, A6.4.2.1, A6.4.2.2, A6.5, A7.2.1, A8.5, A8.6.1, A11.5

b_0	width of the outstanding area of linear members for shear lag purposes. Hollow rectangular profile width. Chord width. Width of sheet between longitudinal stiffeners;	21.1, 21.3.2, 21.3.5, 21.4, 21.5, 34.1.2.5, 42.6, 64.2, 64.7.1, 64.7.2.1, 64.8, 64.9, A7.2.3, A-9-7, A-9-9, A-9-10, A-9-11, A-9-12, A-9-13, A-9-14, A-9-15, A-9-17, A-9-18, A-9-19
b_1	length of the second piece to be joined at the fillet joint. Width of the hollow profile of diagonal or upright 1. Flange width. Size. Distance between stiffeners or between the stiffener and the edge of the equivalent sheet or column;	59.3.4, 64.7.2.1, 64.9, 80.2, A6.4.1, A6.4.2.1, A6.4.2.2, A-9-2, A-9-3, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11, A-9-12, A-9-17, A-9-18, A-9-19
b_{1e}	effective width for exterior flanges for shear lag purposes;	21.3, 21.4
b_2	width of the hollow profile of diagonal or upright 2. Flange width. Size. Distance between stiffeners or between the stiffener and the edge of the equivalent sheet or column;	64.7.2.1, 80.2, A6.4.1, A6.4.2.1, A6.4.2.2, A-9-7, A-9-8, A-9-9, A-9-12, A-9-17, A-9-19
b_3	size. Distance between stiffeners or between the stiffener and the edge of the equivalent sheet or column. Width of the hollow profile of diagonal or upright 3;	80.2, A6.4.1, A-9-12
b_4	size;	80.2
b_b	beam flange width;	62.1.1
b_c	width of support flanges. Width of the compresion part of the element or subpanel;	62.1.1, A6.4.2.1, A6.4.2.2, A6.4.3
$b_{c,loc}$	Width of the compresion part of each subpanel;	A6.4.1
b_e	effective width for interior flanges for shear lag purposes;	21.3, 21.4, 21.6
$b_{e,f}$	effective width of the bending column flange;	62.1.1, 62.3
$b_{e,ov}$	effective width of the covering filler bar at the connection to the covered filler bar;	A-9-7, A-9-17, A-9-19
$b_{e,p}$	effective width of the bending end sheet. Effective punching width;	62.3, A-9-8, A-9-9, A-9-10

b_{ef}	effective width of the beam flange. Effective width of the concrete area under one compressed column flange. Effective width in compresion interior panels;	62.1.1, 62.1.2, 65.2.2, 65.2.5, 73.9.2
b_{eff}	effective filler bar width at the connection to the chord;	64.8, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11, A-9-18, A-9-19
$b_{eff,t,wb}$	effective width of the tension beam web at a bolted joint with end sheet;	61.2.1
b_f	width of the flange leading to the lowest resistance to shear local buckling;	35.5.2.2, 35.6
b_{fal}	total uninterrupted length of the skirt in the front or cover structure;	73.11.3
b_g	length of the transverse stiffener between flanges;	A7.2.4
b_i	width of diagonal or upright i;	64.2, 64.6.1, 64.7.1, 64.7.2.1, 64.8, 64.9, A-9-7, A-9-8, A-9-9, A-9-10, A-9-14, A-9-17, A-9-18, A-9-19
b_j	width of diagonal or upright j;	64.2, 64.6.1, 64.7.1, 64.8, 64.9, A-9-7, A-9-17, A-9-19
b_p	straight width, length of each plate element in uniform section parts formed by plate elements and the according small radius curves. Plate width;	73.5, 73.9.1, 73.9.2, A-9-14, A-9-15
b_s	flange semi-width in closed or hat sections, width in sections Z or C, for bending members of wide flanges in comparison with the depth;	73.7, 73.8
b_w	effective width of the chord web;	A-9-17, A-9-18
c	width or length of part of the cross section. Distance of the anchor from the diagonal tension field. Size. Clearance. Useful depth. Specific heat;	20.3, 35.5.2.2, 35.6, 48.2, 58.9, 59.9.2, 61.6, 65.2.2, 65.2.4, 73.6, A8.5, A8.6.1, A8.7.1
c_a	specific heat in J/(kgºK), variable with temperature (θ_a);	45.1, 48.1
c_p	conventional specific heat;	45.2, 48.3
c_{pd}	design value specific coating heat;	48.2, 48.3

c_{pk}	characteristic value of specific coating heat;	48.3
d	size; Nominal diameter of the bolt, pin or anchoring member. Distance. Diagonal length. Diameter of a bar. Diameter of the circular neoprene support;	18.2.5, 20.3, 29.3, 29.4, 42.6, 58.3, 58.6, 58.9, 60.1.2, 61.1, 61.2, 61.2.1, 61.6, 62.1.4, 62.2.2, 62.4.3, 64.2, 64.4, 66.1, 71.2.3.1, 72.4.3, 73.6, 76.7.1, 76.7.3
d_0	hole diameter. Diameter of hollow circular profile. At welded joints between HCS hollow circular section profiles, chord diameter;	34.1.2.2, 42.6, 58.4, 58.5.2, 58.6, 58.9, 64.2, 64.4, 64.6.1, A-9-1, A-9-2, A-9-3, A-9-4, A-9-5
d_1	diameter of hollow profile of member 1;	64.7.2.1, A-9-1, A-9-4, A-9-5, A-9-7, A-9-9, A-9-15, A-9-17, A-9-19
d_2	diameter of hollow profile of member 2;	64.7.2.1, A-9-1, A-9-7, A-9-8, A-9-9, A-9-15
d_3	diameter of hollow profile of member 3;	A-9-5
d_c	straight part of column web;	62.3
d_i	diameter of hollow profile of member i;	64.6.1, 64.7.1, 64.8, 64.9, A-9-1
d_j	diameter of hollow profile of member j;	64.7.1
d_k	relative displacement between the header and footer of the supports for the storey in question;	50.2
d_{LT}	factor;	35.3
d_m	lower average diameter measured between circumscribed and registered circles at the nut or the head;	58.7
d_p	thickness of protection material. Peripheral diameter in an arc spot weld;	48.2, 48.4, 73.13.5.1
d_s	design diameter of a weld point;	73.13.4
d_t	length of the diagonal subject to an axial tension force N_t;	72.4.3

d_w	diameter of the washer or slot of the bolt or nut head, as applicable. Depth of the web of an I- or H-section chord. Surface diameter of an arc spot weld;	61.2, 64.8, 73.13.5.2
d_{wc}	column web height;	62.1.4, 62.4.3
e	distance between stiffener centres. Distance. Thickness. Distance from the bracket-profile joint weld to its centroid. Eccentricity at the node. Greater of the existing differences between the centroid of the shorter section of the stiffener and of the corresponding part of the support plate and the centroid of the plate or of the gross section of the stiffener alone. Maximum distance from the lower face of the transverse stiffener web to the neutral fibre of the net section of the stiffener + sheet;	35.9.3.1, 58.3, 59.5, 60.3, 61.2, 61.5, 62.2.1, 64.2, 73.13.4, A6.4.2.1, A6.4.3, A7.2.4
e_0	extension of the imperfection (arc deflection) of a element;	22.3.2, 22.3.3, 22.3.4.1, 22.3.4.2, 22.3.5, 22.4, 22.4.1, 22.5, 71.1, 71.2.3
e'_0	scaling applied to the strain set of the critical global buckling mode allowing for the initial eccentricity e_o;	22.3, 22.3.5, 22.4
$e_{0/p}$	warping eccentricity;	42.6
e_1	The end from the centre of the hole in the anchoring member to the adjacent end of any member, measured in the direction of the load transfer. Distance, measured in the direction of the axis of the column, from the upper row of bolts to the end of the column;	42.6, 58.4, 58.6, 62.2.1, 73.13.4
e_2	distance from the centre of the hole in the anchoring member to the adjacent border of any member, measured perpendicularly to the direction of the load transfer;	42.6, 58.4, 58.5.2, 58.6, 73.13.4
e_{max}	maximum distance from the extreme fibre of the stiffener to its centroid;	A7.2.1
e_n	total thickness of the neoprene at the support;	66.1
e_N	displacement of the centroid of the area of the effective section in relation to the centroid of the gross cross section. Positional eccentricity of the neutral fibre of the effective section against the neutral fibre of the gross section subject to a compression axial force;	20.7, 34.1.2.4, 34.7.2.3, 35.3, 35.7.1, 35.7.2, A6.5
f	coefficient of amendment for χ_{LT};	35.2.2.1

f_0	frequency of the first mode of vertical vibration, considering only permanent loads;	38.3
$f_{0.2p,\theta}$	design strength in a fire situation with a strain of 0.2 %.	46.7
f_b	buckling strength of the lateral face of the chord;	A-9-8
f_{bv}	resistance to local buckling by shear tension;	73.10
f_{ck}	characteristic strength of foundation concrete;	65.2.2
f_d	pressure of shaft against concrete at the foundation support joint;	65.3
f_j	base plate pressure against the concrete;	65.2.3, 66.2
f_{jd}	maximum support foundation concrete strength;	65.2.2
f_M	deflection component due to bending;	67.2.1.2
$f_{p,\theta}$	temperature proportionality limit (θ_a);	45.1
f_r	fundamental torsional frequency of the structure;	38.4
f_u	maximum unit load under tension or resistance to tension;	26.2, 26.3, 26.5.2, 27.1, 27.2.1, 27.2.2, 27.2.3, 27.2.4, 27.3. 34.2, 34.4, 58.5.1, 58.5.2, 58.6, 58.7, 59.8.2, 59.10, 60.1.1, 61.3, 61.4, 61.5, 61.6, 73.4, 73.13.4, 73.13.5.1, 73.13.5.2
f_{ub}	tensile strength of steel for bolts and pins;	29.2, 29.4, 58.6, 58.7, 58.8, 65.2.1
f_{ub}	ultimate tensile strength in bolts or pins;	58.1, 58.6, 58.7, 62.1.1, 62.4.3, 76.7
f_{up}	tensile strength of pin steel;	58.9
f_{uw}	ultimate strength of electrode material;	73.13.5.2
f_V	shear deflection component;	67.2.1.2

f_y	yield strength;	20.3, 22.3.2, 24.3.1, 26.2, 26.3, 26.5.2, 27.1, 27.2.1, 27.2.2, 27.2.3, 27.2.4, 27.3. 32.2, 32.3. 34.1.1, 34.2, 34.3, 34.4, 34.6, 34.7.1, 34.7.2.1, 34.7.2.2, 34.7.2.3, 34.7.3, 35.1.1, 35.1.2, 35.1.3, 35.1.4, 35.2.1, 35.2.3, 35.3, 35.5.1, 35.7.1, 35.7.2, 41.2, 42.3, 45.1, 46.1, 46.2, 46.3, 46.5, 46.7, 56.1, 58.2, 58.5.1, 58.9, 60.2.1, 60.3, 61.2, 61.5, 61.6, 62.1.4, 62.4.3, 64.2, 65.2.2, 66.2, A3.3.1, A3.2.2, A5.2, A6.4.2, A6.4.3, A6.5, A7.2.1
$f_{y,0}$	yield strength of steel chord;	64.2, 64.8
$f_{y,red}$	reduced value of steel yield strength;	66.2
$f_{y,\theta}$	effective temperature yield strength (θ_a);	45.1
f_{y0}	yield strength of steel chord;	A-9-1, A-9-2, A-9-3, A-9-4, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11, A-9-17, A-9-18, A-9-19
f_{y1}	steel yield strength of diagonal or upright 1;	A-9-10, A-9-11, A-9-17, A-9-18
f_{ya}	average steel yield strength of cold-formed profile or sheet;	73.4, 73.11.1, 73.11.4
f_{yb}	yield strength of steel for bolts and pins. Yield strength of beam flange steel. Basic steel yield strength of cold-formed profile or sheet;	29.2, 29.4, 46.7, 58.1, 62.1.1, 65.2.1, 73.4, 73.9.2, 73.9.3, 73.10, 73.11.1, 73.11.4

f_{ybp}	yield strength of reinforcement sheets;	61.2
f_{yc}	steel yield strength of support flange;	62.1.1, 62.1.2
f_{yf}	steel yield strength of flanges. Steel yield strength of compressed flange;	35.5.2.2, 35.6, 35.8
f_{yi}	steel yield strength of diagonal or upright i;	64.8, A-9-7, A-9-9, A-9-17, A-9-19
f_{yj}	steel yield strength of diagonal or upright j;	A-9-7, A-9-8
f_{yk}	characteristic steel yield strength;	32.1, A7.2.4, A-9-11
f_{yp}	yield strength of pin steel. Yield strength of sheet steel;	58.9, A-9-14
f_{yw}	yield strength of flange steel;	35.5.2, 35.5.2.1, 35.5.2.2, 35.6, 35.7.2, 35.9.3.3, 35.9.3.5
f_{ywb}	steel yield strength of the tension beam web at a bolted joint with end sheet;	64.2.1
f_{ywc}	steel yield strength of non-stiffened column web;	62.1.4
g	gap between the feet of two contiguous brace elements in K, N or similar joints, measured along the chord face in the plane of the joint, regardless of the thickness of the weld (negative values of g represent coating q):	42.6, 64.2, 64.6.1, 64.7.1, 64.8, A-9-1, A-9-6, A-9-9, A-9-15
g_1	distance from the outer face of the diagonal or upright to the face of the chord;	64.4
g_2	distance from the inner face of the diagonal or upright to the face of the chord;	64.4
h	size. Height. Structure height. Section depth. Effective support height. Distance. Shoe thickness. Belt thickness. Distance between floors. Maximum operator-accessible distance in a narrow space;	20.3, 22.3.1, 34.5, 34.7.2.1, 35.1.2, 35.2.2, 35.2.2.1, 35.2.3, 37.2.2, 48.2, 61.1, 61.2.1, 61.3, 62.1.5, 65.2.3, Art. 67, 73.5, 73.6, 73.11.3, 80.2, 80.3, 92.2
h_0	depth of hollow rectangular profile. Chord depth. Distance between centroids of compound member chords;	42.6, 64.2, 64.7.1, 71.2.3, 71.2.3.1, 71.2.3.2, A-9-8, A-9-9, A-9-10, A-9-13, A-9-18, A-9-19

h_1	distance of the straight part of the support web. Height between floors. Depth of brace member 1;	62.1.2, 80.3, A-9-2, A-9-3, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11, A-9-12, A-9-15, A-9-17, A-9-18, A-9-19
h_2	distance between floors. Depth of brace member 2;	80.3, A-9-7, A-9-8, A-9-9, A-9-12, A-9-15, A-9-17, A-9-19
h_3	distance between floors. Depth of brace member 3;	80.3, A-9-12
h_b	beam depth;	62.4.2
h_c	support depth;	62.1.4, 62.4.2
h_{ef}	effective web height;	62.1.2, 62.2, 62.3
h_{eq}	weighted average of the height of openings in all walls;	A8.6.1, A8.4.6
h_i	depth of member i;	64.2, 64.6.1, 64.7.1, 64.8, 64.9, A-9-7, A-9-8, A-9-14, A-9-17, A-9-18, A-9-19
h_j	depth of diagonal or upright j;	64.2, 64.6.1, 64.8, A-9-7, A-9-17, A-9-19
h_k	distance between floors;	50.2
h_p	Storey height in question;	23.2.1
h_r	distance between bolt row r and the centre of compression;	62.3
h_s	stiffener height including sheet thickness;	A7.2.4
h_w	web distance. Distance between average web end points, measured vertically for cold-formed sections or sheets. Height of the rib;	34.5, 34.7.1, 34.7.2.1, 35.5.1, 35.5.2, 35.5.2.1, 35.5.2.2, 35.6, 35.7.1, 35.8, 35.9.3.1, 35.9.3.3, 35.9.3.5, 35.9.4, 73.10, 73.11.3, A6.4.2.1
$\dot{h}$	Heat flux received through radiation in the surface unit exposed to fire at ceiling height;	A8.6.2

$\dot{h}_1$, $\dot{h}_2 \ldots$	thermal flows received by surface unit exposed to fire at ceiling height for different, separately-located fires;	A8.6.2
$\dot{h}_{net}$	net heat flow incident upon the member surface unit exposed to fire;	A8.2.2, A8.6.2, A8.7.1
$\dot{h}_{net,c}$	thermal flow convection component by surface unit;	A8.2.2
$\dot{h}_{net,d}$	net heat flow by unit of area, in W/m^2;	48.1
$\dot{h}_{net,r}$	net heat flow radiation component by surface unit;	A8.2.2
$\dot{h}_{tot}$	total thermal flow calculated as the sum of $\dot{h}_1$, $\dot{h}_2 \ldots$ etc.;	A8.6.2
i	radius of gyration;	35.1.3, 61.2, 62.3, A6.4.3
$i_{f,z}$	radius of gyration of compresion flange equivalent to the weak axis of the section;	35.2.3
i_{mvn}	minimum radius of gyration;	Art. 70, 70.5, 71.1
k	Factor. Coefficient of use;	35.8, 57.4, 73.4, 91.2.2.5, A8.6.1
$k_{0.2p,\theta}$	quotient between high temperature resistance and yield strength at 20 °C;	46.7
k_1	increase factor for nominal stress ranges in triangular structures for taking into account the effect of secondary moments. Adaptation factor for non-uniform temperature distributions in the cross section;	42.5, 46.4
k_2	adaptation factor for non-uniform temperature distributions along the beam;	46.4
k_{at}	stiffness of each row of bolts in the base;	65.2.5
k_b	stiffness of one row of bolts under tension. Conversion factor;	62.3, A8.5
k_c	correction factor;	A8.5
k_c	slenderness correction factor for considering the distribution of moments between bracing points. Stiffness provided by the concrete and base plate under compression;	35.2.2.1, 35.2.3, 65.2.5
$k_{E,\theta}$	ratio between the elastic modulus in the linear phase of the stress-strain diagram for temperature (θ_a) and the elastic modulus at 20 °C;	45.1, 46.3, 46.6

$k_{eff,r}$	effective factor of stiffness for row of bolts r;	62.3
k_{eq}	equivalent coefficient of stiffness;	62.3
k_F	local buckling coefficient;	35.6
k_f	Bending stiffness of column flange, corresponding to one row of bolts;	62.3
k_{fl}	correction factor;	35.2.3
k_g	factor;	64.6.2, A-9-1
k_h	reduction factor for the height h of the structure;	22.3.1
k_j	stiffness coefficient for each basic component;	62.3
k_m	factor;	22.3.1, 22.4, 22.4.1, A-9-10
k_n	factor;	64.7.3, A-9-7, A-9-8, A-9-9, A-9-11
k_p	bending stiffness of end sheet, corresponding to one row of bolts. Factor. Stiffness of bending base plate;	62.3, 64.6.3, 65.2.5, A-9-1, A-9-2, A-9-3, A-9-4,
$k_{p,\theta}$	ratio between the proportionality limit for temperature (θ_a) and the yield strength at 20 °C;	45.1
k_s	reduction factor for resistance to fatigue due to size. Factor used to calculate bolt slip resistance;	42.6, 58.8
k_{sh}	coefficient for calculating steel temperature in unprotected members;	48.1
k_{wc}	compression column web stiffness;	62.3
k_{wc}	coefficient which takes into account the maximum compression tension $\sigma_{n,Ed}$ of a support web, deriving from the axial force and design moment to which the support is subjected at its joint to the beam;	62.1.2
k_{wt}	Tension stiffness web;	62.3
k_{wv}	shear column web stiffness;	62.3
k_{zz}	interaction coefficient;	35.3
$k_{z\gamma}$	interaction coefficient;	35.3

k_{γ}	non-dimensional coefficient;	22.3.5
$k_{\gamma,\theta}$	ratio between the effective yield strength for temperature (θ_a) and the yield strength at 20 °C;	45.1, 46.2, 46.3, 46.4, 46.6
$k_{\gamma,\theta,com}$, $k_{E,\theta,com}$	correction coefficients obtained at 45.1 with the maximum temperature of the compresion zone of the section ($\theta_{a,com}$) at time (t) of the fire process in question;	46.5
$k_{\gamma,\theta,i}$	ratio between the effective yield strength for temperature (θ_a) and the yield strength at 20 °C for the partition of a section in members of area A_i;	46.2
$k_{\gamma,\theta,V}$	value corresponding to θ_V;	46.4
$k_{\gamma y}$	interaction coefficient;	35.3
$k_{\gamma z}$	interaction coefficient;	35.3
k_{σ}	panel local buckling coefficient. Transverse buckling or warping coefficient;	20.7, 40.2, 73.9.2
${}_{\sigma}k_{,p}$	stiffened panel local buckling coefficient;	A6.4.2.1
${}_{\tau}k$	shear local buckling coefficient;	35.5.1, 35.5.2.1, 35.9.3.3, 40.2, 73.10
l	length;	22.3.4.1, 22.3.4.2, 42.6, 60.2, 66.2, A7.2.4
l_{eff}	effective length of the area of concrete under a compressed column flange;	65.2.2, 65.2.5
l_{eff}	effective length of an fillet weld or of a bolted joint;	61.2, 62.2.1
l_o	buckling length;	22.3.4.2
l_p	flange plate length;	A-9-14, A-9-15
l_y	effective load length;	35.6
m	number of compresion member alignments in the plane buckling in question. Mass by unit of length of the structure in Kg/m. Inverse the inclination of the fatigue resistance curve. Distance. Corresponding distance of the bolt to the line forming the corresponding plastic hinge. Combustion factor;	22.3.1, 38.4, 42.2, 58.4, 61.2, 62.2.1, 62.3, 65.2.5, A8.4.1, A8.4.5, A8.4.6, A8.7.1

m(z)	law of distribution of outer torsional moments along the longitudinal axis of the symmetrical single-cell box beam;	A3.3.1
m_1	non-dimensional coefficient;	35.6
m_2	non-dimensional coefficient;	35.6
$\dot{m}$	rate of variation of gas mass;	A8.7.1
$\dot{m}_{fi}$	mass of products generated by pyrolysis in the unit of time;	A8.7.1
m_i	parameter;	A11.4.1
$\dot{m}_{in}$	mass of air entering through openings in the specified unit of time;	A8.7.1
$\dot{m}_{out}$	mass of gas exiting through openings in the specified unit of time;	A8.7.1
n	whole number. Factor. Number of extended holes in any diagonal or zigzag line through the member or part thereof. Number of joint bolts. Distance. Number of triangulation planes. Number of batten plate planes. Number of folds in a 90° section. Value $(\sigma_{0,Ed} / f_{y,0}) / \gamma_{M5}$ used for SHR chords;	21.1, 21.6, 34.1.2.2, 34.7.2.1, 58.5.1, 58.6, 58.8, 60.1.2, 60.2.2, 61.2, 61.3, 70.4, 71.2.3.1, 71.2.3.2, 73.4, 80.3, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11
n_b	number of rows of bolts;	61.2
n_i	number of load cycles (i = 1, 2...n). Parameter;	42.6, A11.4.1
n_p	value $(\sigma_{p,Ed} /f_{y,0}) / \gamma_{M5}$ used for SHC chords;	A-9-1, A-9-2, A-9-3, A-9-4
n_{pl}	factor;	35.3
p	spacing between centres of two consecutive holes measured perpendicular to the axis of the element. In K, N or similar joints, length of the intersection of the diagonal or upright overlapped with the chord, measured along its face on the plane of the joint;	34.1.2.2, 64.2
p(z)	vertical load applied following the longitudinal axis of the box beam;	A3.3.1
p_1	spacing between centres of anchoring members in line with the load transfer direction;	42.6, 58.4, 58.6, 73.13.4

$p_{i,1}$ a $p_{i,4}$	percentages;	A11.4.3.1 a A11.4.3.4
$p_{i,5}$	factor;	A11.4.3.5
p_2	distance between adjacent rows of anchoring members measured perpendicular to the load transfer direction;	42.6, 58.4, 58.6, 73.13.4
q	equivalent stabilisation force by unit of length. In K, N or similar joints, theoretical overlap length, measured along the face of the chord on the plane of the joint. Uniformly distributed lateral load;	22.4.1, 42.6, 64.2, A7.2.1, A-9-1
q_{fd}	design value of the fire load;	A8.4.1, A8.6.1
q_{fk}	Characteristic fire load density performance per floor area unit;	A8.4.1
q_{td}	design value of the system of transverse forces equivalent to the initial curve in compresion elements. Design value of the load density to fire in relation to the total fire protection area A_t;	22.3.3, A8.6.1
r	root radius. Radius of curvature of compresion flange. Mass radius of gyration. Bolt row number. Radius of support sphere. Horizontal distance between vertical axis of fire and point of ceiling for which the heat flow is calculated;	34.5, 35.8, 38.4, 42.6, 61.2, 61.2.1, 61.5, 61.6, 62.3, 66.2, A8.6.2, A-9-17, A-9-18
r_0	flange-web fillet radius of the I or H section chord;	64.9
r_c	support flange-web fillet radius, assuming this is rolled;	62.1.1, 62.1.2, 62.2.1
$r_{p,ef}$	effective thermal resistivity;	45.2, 48.4
$r_{p,ef,d}$	design effective thermal resistivity of the coating;	Art. 47, 48.2, 48.3
$r_{p,ef,k}$	characteristic effective thermal resistivity of the coating;	Art. 47, 48.2, 48.3, 48.4
s	arc length. Distance between centres of two consecutive holes measured parallel to the axis of the member. Value obtained as $s=S_p/r_{p,ef}$. Side length of one section of a fillet weld chord. Distance between splices in single supports interconnected with sheet linings. Separation between belts. Distance. Thickness of layer exposed directly to fire;	22.3.4.1, 34.1.2.2, 48.4, 59.8, Art. 70, 73.11.3, A7.2.1, A8.6.1
s_1	separation between slots;	59.5
s_2	separation between rows of slots;	59.5

s_d	Chord extension length in cold-form sections and sheets;	73.10
s_e	length of the load distribution area at the flange-web point of contact, in the case of application of loads on the plane of the web of a section;	21.6
s_{lvm}	limit thickness;	A8.6.1
s_p	straight length of the greater section of a web in cold-form sections and sheets;	73.10
s_s	length of the web cover plate load application area, in the case of application of loads on the plane of the web of a section. Length of the web on which the load is applied;	21.6, 35.6
s_w	total straight length of a web in cold-form sections and sheets;	73.10
t	element thickness. Fire time duration. Thickness of the thinnest piece to be joined. Thickness of each of the neoprene support sheets;	18.2.5, 20.7, 21.4, 27.1, 27.2.1, 27.2.2, 27.2.3, 27.2.4, 28.1, 32.3, 32.3, 34.1.2.2, 34.1.2.5, 34.5, 34.7.2.1, 35.9.1, 40.2, 42.6, Art. 46, 46.3, 46.4, 46.5, 46.6, 46.8, 46.8.1, 46.8.2, Art. 47, 48.1, 48.4, 58.4, 58.5.2, 58.6, 58.7, 59.3.4, 59.3.6, 59.5, 59.9.2, 60.3, 61.2, 61.5, 61.6, 62.4.3, 64.2, 65.2.2, 65.2.5, 66.1, 73.4, 73.6, 73.7, 73.9.2, 73.10, 73.11.3, 73.13.4, 73.13.5.1, 73.13.5.2, 76.7.3, A3.2.2, A3.3.1, A6.4.1, A6.4.2.1, A6.4.2.2, A6.4.3, A8.3.1, A8.3.2, A8.3.3, A8.4.6, A8.5, A8.6.1, A8.7.1, A-9-13
t_0	hollow profile thickness. Weld hollow profile thickness;	42.6, 64.2, 64.4, 64.6.1, 64.7.2.1, A-9-1, A-9-2, A-9-3, A-9-4, A-9-7, A-9-

		8, A-9-9, A-9-10, A-9-11, A-9-14, A-9-15, A-9-19
t_1	thickness of the second piece to be joined at the fillet joint. Thickness of the reinforcement sheet. Thickness of diagonal or upright 1;	59.3.4, 62.2.1, 64.2, 64.4, 64.6.1, 64.7.1, 64.9, 73.13.4, A-9-1, A-9-2, A-9-9, A-9-10, A-9-11, A-9-15, A-9-17, A-9-18
t_2	thickness of brace member 2. Lining thickness;	64.2, 64.6.1, 64.7.1, 64.9, 73.13.5.2, A-9-1, A-9-9, A-9-15, A-9-17
t_a	steel thickness in a small tube in relation to its perimeter;	48.1
t_{bp}	thickness of reinforcement sheets;	61.2
t_c	chord reinforcement thickness;	42.6
t_{cor}	design thickness of steel profile or light steel structure;	73.3
t_d	thickness of diaphragm sheet;	A3.3.3
$t_{e,d}$	equivalent standardised fire exposure time;	A8.2.1, A8.5
t_f	thickness of compresion sheet panel. Flange thickness. Lining thickness;	20.3, 34.5, 34.7.2.1, 35.1.2, 35.2.3, 35.5.2.2, 35.6, 48.1, 58.6, 59.8.2, 61.2, 61.5, 61.6, 64.2, 64.8, 65.2.2, A-9-17, A-9-18
t_{fb}	thickness of connected beam flange;	61.2.1, 62.1.2, 62.1.4, 62.1.5
t_{fc}	thickness of support flange;	62.1.1, 62.1.2, 62.1.4, 62.3
$t_{fi,nom}$	duration expressed in standard minutes of fire;	A8.2.1, A8.5
$t_{fi,requ}$	standardised time of resistance to fire required of the structure;	Art. 46, 48.3, 48.4, A8.1
t_g	working life actually considered in the design for the structure within the ranges included in this Code;	A11.5

$t_{g,mvn}$	working life established in section 5.1 of this Code;	A11.5
t_i	wall thickness of member I (i =1,2 or 3). Wall thickness of the hollow profile of diagonal or upright i;	42.6, 64.6.1, 64.7.1, 64.8, 64.9, A-9-7, A-9-8, A-9-10, A-9-17, A-9-19, A-9-19
t_j	wall thickness of the hollow profile of diagonal or upright j;	A-9-7, A-9-17, A-9-19
t_{lvm}	fire development time limit;	A8.6.1
t_{max}	maximum temperature time;	A8.6.1
t_{mc}	nominal thickness of steel profile or light steel structure;	73.3
t_{min}	minimum sheet thickness. Thickness of thinnest piece to be joined;	58.7, 59.3.2, 59.3.7, A7.2.4
t_{nom}	galvanised steel thickness;	73.3, A8.1,
tol	tolerance;	73.3
t_p	end sheet thickness. Flange plate thickness;	62.3, A-9-13, A-9-14, A-9-15
t_r	stiffener thickness;	62.1.3
t_{sup}	thickness of affected sheet in terms of sheet metal bolt extraction capacity. Thickness of support sheet at an arc spot weld;	73.13.3, 73.13.5.2
$t_{ti,d}$	design value of standardised fire resistance of members;	A8.5
t_w	flange thickness;	20.3, 21.6, 34.5, 34.7.1, 34.7.2.1, 35.1.2, 35.5.1, 35.5.2, 35.5.2.1, 35.5.2.2, 35.6, 35.7.2, 35.8, 35.9.3.1, 35.9.3.3, 35.9.3.5, 59.8.2, 61.2, 61.4, 61.5, 61.6, 62.1.3, 62.1.4, 62.4.3, 64.8, 64.9, 65.2.2, A-9-17, A-9-18, A-9-19
t_{wb}	flange thickness of the tension beam web at a bolted joint with end sheet;	61.2.1

t_{wc}	thickness of column web;	62.1.1, 62.1.2, 62.1.3, 62.1.4, 62.3
t_{α}	time required to reach a heat release rate of 1MW.	A8.4.6
u	total horizontal deflection of the building or structure of height H. Flange bending in elements subject to wide flange bending in comparison with the depth. Material dampness, as a dry weight percentage;	37.1, 37.2.2, 73.7, A8.4.4
u_i	relative horizontal deflection between floor slab heights, on each level or storey of height H_i;	37.1, 37.2.2
w	angle profile gage. Factor which takes into account interaction with the column web shear;	61.3, 62.1.2
w_0	initial sinusoidal imperfection;	A7.2.1
w_1	initial deflection under the entirety of permanent loads acting on the structure;	37.1
w_2	deferred component of deflection under permanent loads;	37.1
w_3	deflection due to the action of overloads, under the relevant combination of actions;	37.1
w_{activa}	active deflection;	37.1, 37.2.1
w_c	inverse deflection of structural element;	37.1
w_{el}	elastic deflection;	A7.2.1
w_f	coefficient of ventilation;	A8.5
w_{max}	total apparent deflection discounting the camber;	37.1, 37.2.1
w_{tot}	total deflection;	37.1
z	transverse distance between the section in question and the flange-web point of contact, immediately after load application, for loads situated on the plane of the web of a section. Lever arm. Distance to the neutral fibre. Height along the flame axis;	21.6, 62.1.4, 62.1.5, 65.2.2, 65.2.5, 73.7, A8.6.2
z´	vertical position of the virtual heat source;	A8.6.2
z_0	virtual origin of the axis of the flames;	A8.6.2

z_i	position of the result of F_{zi} stresses (mechanical arm of such force) on member surface A_i;	46.4

A1.1.3 Greek Capitals

Δ	difference. Increment. Deviation (tolerances). Eccentricity. Tapering. Collapse. Clearance;	16.1, 20.7, 32.3, 80.2, 80.3, 80.3.1, 80.4
Δ 1	tapering;	80.3
Δ 2	tapering;	80.3
Δ_a	geometric deviation;	16.1
ΔL	temperature(θ_a) induced expansion of a member of length L;	45.1
ΔM	additional moment	20.7. 34.3
ΔM_{Ed}	additional bending moment in class 4 cross-sections subject to axial compression force due to displacement of the axis of effective area A_{ef} related to the gross cross section;	34.1.2.4, 35.3
ΔN_{st}	increment of axial compression force in the transverse stiffener in order to account for deviation forces;	A7.2.1
Δt	time increment in seconds;	48.1, 48.2
$\Delta T\varepsilon_{cf}$	term for the rate of deformation for checking the fracture toughness of a steel;	32.3
Δt_p	Retardation time of protection materials with a permanent humidity content;	45.2, 48.2
ΔT_r	effect of loss due to radiation of steel for checking fracture toughness;	32.3
$\Delta T\varepsilon_{cf}$	cold steel forming term for checking fracture toughness;	32.3
$\Delta\theta_{a,t}$	member increase in steel temperature;	48.1
$\Delta\theta_{g,t}$	increment of $\theta_{g,t}$ during Δt;	48.2
$\Delta\sigma$	direct nominal stress ranges;	42.2, 42.3, 42.6
$\Delta\sigma_{C,red}$	reduced fatigue resistance reference value;	42.2, 42.6

$\Delta\sigma_c$; $\Delta\tau_c$	value of resistance to fatigue where N_c = 2 million cycles;	42.2, 42.6
$\Delta\sigma_D$; $\Delta\tau_D$	stress range fatigue limit for a constant increase at a number of cycles N_D;	42.2, 42.6
$\Delta\sigma_i$; $\Delta\tau_i$	direct and shear component stress ranges in the n^{th} load cycle;	42.3
$\Delta\sigma_L$; $\Delta\tau_L$	damage threshold for stress ranges in cycle number N_L;	42.2, 42.6
$\Delta\tau$	nominal shear stress ranges;	42.2, 42.3, 42.6
Θ	angle between filler bar and chord;	42.6
Θ_g	temperature of gas in proximity of element exposed to fire;	A8.2.2, A8.2.3, A8.3.1, A8.3.2, A8.3.3, A8.6.1
Θ_m	element surface temperature;	A8.2.2, A8.6.2
Θ_{max}	maximum temperature during heating phase;	A8.6.1
Θ_r	effective fire radiation temperature;	A8.2.2
Θ_z	temperature of crest along its vertical axis of symmetry;	A8.6.2
ΣM_i	total bending moment affecting the section, sum of "i" individual loads;	21.3.3
Φ	angle. Value for determining the reduction coefficient χ. Form factor;	22.4.1, 35.1.2, 46.3, 46.6, A8.2.2, A.8.6.2
Φ_{Cd}	Joint rotation capacity. Rotation capacity at a welded, non-stiffened support-beam joint;	57.2, 62.4.2
Φ_{LT}	value for determining the reduction coefficient χ_{LT};	35.2.2, 35.2.3, 35.2.2.1
$\Phi_{LT,\theta,com}$	value for determining the reduction coefficient $\chi_{LT,fi}$;	46.5
Ψ	non-dimensional stress or strain coefficient. Ratio of segment-end moments;	20.3, 20.7, 35.2.2.1, 35.3, 73.9.2, 73.10, A6.4.2.1, A6.4.2.2
Ψ'_{el}	elastic reduction coefficient of the effective width of stiffened flanges due to shear lag;	21.4, 21.5
Ψ'_{ult}	reduction coefficient of the effective width of flanges in the elastic-plastic range due to shear lag, for tension flanges with longitudinal stiffeners;	21.5

Ψ_1	coefficient corresponding to the frequent value of a variable action;	42.3
$\Psi_{1,1} Q_{k.1}$	representative frequent value of the determinant variable action;	13.2, Art. 44
$\Psi_{2,.i} Q_{k.i}$	semi-permanent representative value of variable actions acting simultaneously with the determinant variable action and the accidental action, or with the seismic action;	13.2, Art. 44
Ψ_{el}	elastic reduction coefficient of the effective width of flanges due to shear lag;	21.3, 21.4
$\Psi_{el,i}$	elastic reduction coefficient of the effective width corresponding to individual load “i”;	21.3.3
Ψ_i	action reduction factor. Factor;	Sections. 11 and 12, A8.4.1, A8.4.3
$\Psi_{o.i} Q_{k.i}$	representative value of the combination of variable actions acting simultaneously with the determinant variable action;	13.2
Ψ_{ult}	reduction coefficient of the effective width of flanges in the elastic-plastic range due to shear lag, for non-stiffened flanges;	21.5
Ω	double the interior area of the trapezium forming the cross section of the box beam;	A3.3.1

A1.1.4 Greek Lower Case

α	part of a compressed cross section. Factor. Imperfection factor;	20.3, 20.7, 21.4, 22.3.5, 32.2, 32.4, 34.1.2.5, 34.7.2.1, 35.1.2, 35.5.2.1, 46.5, 46.6, 57.2, 58.6, 59.8.2, 61.2, 62.2.1, 71.2.3.1, 72.1.1, 92.2, A6.4.2.1, A6.4.3, A8.6.2
α_b	factor;	65.2.1
α_c	convection heat transfer factor for standard fire;	48.1, A8.2.2, A8.3.3, A8.4.6, A8.7
α_{cr}	coefficient which requires the multiplication of design loads to produce the elastic instability of the general buckling model of the structure;	23.2, 23.2.1, 24.2, 24.3
α_e	factor;	A6.4.3

α_h	factor;	35.3
α_i	factor;	70.4, A11.4.1
α_{LT}	imperfection factor;	35.2.2, 35.2.3, 35.2.2.1
α_s	factor;	35.3
α_u	load multiplier factor for collapse situations;	50.2
α_y	load multiplier factor for initial plastification;	50.2
α_θ	linear expansion factor;	45.1
β	non-dimensional factor. Reliability index. Angle. List of dimensions between a diagonal and an upright and the corresponding chord. Buckling coefficient. Ratio between the average diameter or width of filler bars and the diameter or width of the chord;	21.3.2, 32.2, 34.1.2.5, 34.7.2.1, 35.2.2.1, 58.6, 60.1.2, 61.5, 62.2.2, 62.3, 64.2, 64.7.1, 64.9, 70.3, 70.4, 72.4.1, 72.4.3, A5.2, A-9-1, A-9-2, A-9-3, A-9-4, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11
β'	non-dimensional factor;	21.4, 34.1.2.5
β_1	non-dimensional factor;	59.8.1, 62.3
β_2	non-dimensional factor;	58.5.2, 59.8.1, 62.3
β_3	non-dimensional factor;	58.5.2
β_{50}	reliability index for a reference period of 50 years;	5.1.1.1
$\beta_{A,c}$	factor;	A6.4.2, A6.4.3
β_f	non-dimensional factor;	58.6
β_i	buckling factor for the load Pi, considered individually. Factor;	70.4, A11.4.1
β_j	non-dimensional factor that depends on the specifications of the levelling mortar poured for	65.2.2

	laying the base plate;	
β_w	steel correlation coefficient;	59.8.2, 59.10, 60.1.1, 60.2.1, 61.4, 61.6
γ	density. Relation between the size of a chord and double its thickness. Angle distortion. Factor;	32.4, 64.2, 66.1, A6.4.2.1, A-9-1, A-9-4, A-9-9, A-9-17
γ_A	partial safety factor of accidental action.	12.1,
γ_f	partial safety factor for an action. Specific weight. Angle distortion;	Art. 12,
γ_{Ff}	partial increase factor for stress ranges;	42.2, 42.3
γ_G	permanent action partial safety factor.	12.1, 12.2
γ_{G*}	permanent action partial safety factor of a non-constant value.	12.1, 12.2
γ_{i*}	coefficient;	A11.4.1
γ_M	material strength reduction factor;	15.2, 32.1, 32.2, A11.4.3.3
γ_{M0}	partial coefficient for the strength of cross sections;	15.3, 34.1.1, 34.2, 34.3, 34.4, 34.5, 34.6, 34.7.1, 34.7.2.1, 34.7.2.2, 34.7.2.3, 35.5.2.2, 35.7.1, 35.7.2, 46.2, 46.4, 58.2, 58.5.1, 58.9, 60.3, 61.2, 61.2.1, 62.1.1, 62.1.2, 62.1.4, 65.2.2, 73.9.2, 73.10, 73.11.1, 73.11.2, A5.2, A6.3, A6.5, A7.2.4
γ_{M1}	partial coefficient for the resistance to instability of structural members;	15.3, 35.1.1, 35.2.1, 35.2.3, 35.3, 35.5.2, 35.7.2, 35.9.3.3, 35.9.3.5. 46.5, 62.1.2, A7.2.1
γ_{M2}	partial coefficient for the failure strength of tensile cross sections. Also for joint strength;	15.3, 34.2, 34.4, 58.5.1, 58.6, 58.7, 58.9, 59.8.2, 59.10, 60.1.1,

		60.2.1, 61.3, 61.4, 61.5, 61.6, 65.2.1, 73.13.4, 73.13.5.1, 73.13.5.2
γ_{M3}	partial coefficient for the slip resistance of joints with pre-loaded bolts;	15.3, 58.8
γ_{M5}	partial safety factor for strength in joints between tubular section parts;	64.1, A-9-1, A-9-2, A-9-3, A-9-4, A-9-7, A-9-8, A-9-9, A-9-10, A-9-11, A-9-14, A-9-17, A-9-18, A-9-19
γ_{Mf}	partial fatigue resistance coefficient for detail categories $\Delta\sigma_c$; $\Delta\tau_c$, when used as a fatigue resistance value;	42.2, 42.4
γ_{Mfi}	partial strength coefficient for steel in a fire situation;	45.1, 46.1, 46.3, 46.4, 46.5, Art. 47
γ_p	partial coefficient considering the fire protection system;	Art. 47, 48.3, 48.4
γ_Q	variable partial safety factor.	12.1, 12.2
γ_q	partial safety factor which takes into account foreseeable consequences of fire;	A8.4.1, A8.5
δ	relative displacement between the top and bottom ends of the support. Factor:	66.1, A6.4.2.1
$\delta_{H, Ed}$	relative horizontal displacement of the upper level of the storey in relation to its lower level;	23.2.1
δ_{ni}	coefficient which takes into consideration the different active measures for fire fighting i, sprinklers, detection and automatic fire fighter alarm transmission (i=1, 2 or 3).	A8.4.1
δ_q	bracing system deflection in the stabilisation plan;	22.4.1
δ_{q1}	partial factor which takes into consideration the activation risk of fire due to type of occupancy;	A8.4.1
δ_{q2}	partial factor which takes into consideration the activation risk of fire due to sector sizes;	A8.4.1, A8.4.6
$\lvert\delta_x\rvert$	maximum local deflection along the member;	35.3
ε	warping;	20.3, 32.2, 35.1.3, 35.2.3, 35.3, 35.5.1, 35.5.2.1, 35.5.2.2, 35.6, 35.9.1, 45.1, 46.1, 62.1.4, 62.4.3,

		73.9.2, 73.11.1, A3.2.2, A3.3.1
$\varepsilon_{c,Ed}$	design maximum compression panel strain;	20.7
ε_{cf}	percentage of permanent strain produced by the cold forming of material;	32.3
ε_{cmax}	maximum single strain on the most compresion edge of the panel;	19.5.1, 19.5.2
ε_{cr}	ideal critical panel local buckling strain;	19.5.1, 20.7
ε_{cu}	limit strain for compresion steel elements;	19.5.1
ε_{f}	flame emissivity;	48.1, A8.2.2, A8.6.2
ε_{m}	member surface emissivity;	A8.2.2, A8.6.2
ε_{max}	strain under maximum load;	26.3, 26.5.2
$\varepsilon_{p,\theta}$	strain in relation to the temperature proportionality limit (θ_a), $f_{p,\theta}$;	45.1
ε_{res}	resultant emissivity for carbon steel surfaces;	48.1
ε_{tu}	limit strain for tensile steel elements;	19.5.1
ε_{u}	ultimate strain;	26.3, 26.5.2
$\varepsilon_{u,\theta}$	ultimate strain for the temperature (θ_a) in the stress-strain diagram;	45.1
ε_{y}	strain corresponding to the yield strength of steel;	19.5.1, 20.3, 26.3
$\varepsilon_{y,\theta}$	strain corresponding to the effective yield strength for temperature (θ_a), $f_{y,\theta}$;	45.1
$\dot{\varepsilon}$	rate of strain;	32.3
$\dot{\varepsilon}_o$	rate of strain reference value;	32.3
η	coefficient which allows for the consideration of additional plastic strength offered by hardening through the deformation of material. Parameter. Relation between the average diameter or width of filler bars and the diameter or width of the chord;	34.5, 35.5.1, 35.5.2.1, 35.9.3.5, 57.5, 64.2, A-9-2, A-9-3, A-9-10, A-9-11
η_0	degree of use;	46.8.1

η_1	coefficient of distribution;	A5.2
η_2	coefficient of distribution;	A5.2
η_{cr}	shape of the elastic critical buckling mode of the structure;	22.3.5
η_{inic}	amplitude of the single imperfection of the elastic critical buckling mode;	22.3.5
θ	temperature of a member. Factor. Angle. Angle between filler bar and chord;	Art. 46, 46.2, 46.6, 48.3, 50.2, 61.6, 64.1, 64.2, A7.2.1, A-9-11, A-9-8, A-9-13
θ_a	temperature reached by steel. Rotation on the near edge of the beam;	45.1, 46.7, 48.4, A5.2
$\theta_{a,com}$	maximum steel temperature in the compressed flange of the section;	46.5
$\theta_{a,cr}$	critical temperature value in accordance with 46.8, 46.8.1 and 46.8.2	Art. 46, 46.8, 46.8.1, 46.8.2
$\theta_{a,m\alpha x}$	maximum steel temperature in the section;	46.5
$\theta_{a,t}$	uniform temperature reached by steel after a standard fire of duration t;	Art. 46, 46.3, 46.4, 46.8, 46.8.2, 48.1, 48.2, 48.4
θ_b	rotation on the far edge of the beam;	A5.2
$\theta_{g,t}$	gas temperature;	48.1, 48.2
θ_i	angle between filler bar and chord (i=1, 2 or 3). Temperature of a member of area A_i resulting from the partition of a section;	64.2, 46.2, A-9-1, A-9-4, A-9-5, A-9-7, A-9-8, A-9-9, A-9-12, A-9-15, A-9-17, A-9-19,
$\theta_{m,t}$	member surface temperature;	48.1
θ_V	average temperature of the effective shear cross-section (A_V) used in room temperature calculations, in accordance with 34.5;	46.4
κ	factor;	A-9-13
λ	slenderness. Thermal conductivity;	22.3.2, 71.2.3.2, 73.11.2, A8.6.1
λ_1	factor;	61.2

λ_2	factor;	61.2
λ_6	parameter;	A11.4.3.6
λ_a	thermal conductivity in W/(mºK), variable with temperature (θ_a);	45.1
λ_E	slenderness value for determining relative slenderness;	35.1.3, 35.2.3
λ_{ef}	effective slenderness;	72.3
$\lambda_{1i}, \lambda_{2i},\ldots \lambda_{5i}$	parameters;	A11.4.3.1, , A11.4.3.2, A11.3.4.3, A11.3.4.4, A11.4.3.5
λ_o	slenderness of a column, considered hinged;	65.2.5
λ_{ov}	overlap ratio as a given percentage;	64.2, 64.7.1, A-9-7, A-9-19
$\lambda_{ov,lim}$	limit overlap ratio as a given percentage;	64.6.1, 64.8
λ_p	conventional thermal conductivity in W/(mºK). Thermal conductivity of the fire protection system;	45.2, 48.3
λ_{pk}	characteristic thermal conductivity of the fire protection system;	48.3
λ_v	slenderness for buckling around the axis of minimum inertia;	72.3
λ_y	slenderness for buckling around the y-y axis, parallel to the flanges;	72.3
λ_z	slenderness for buckling around the z-z axis, parallel to the flanges;	72.3
$\bar{\lambda}$	relative slenderness;	22.3.5. 24.3.1, 35.1.2, 35.3, 46.3, 61.6, 73.11.2, 73.11.3, Annex 4
$\bar{\lambda}_c$	relative column slenderness;	A6.4.3
$\bar{\lambda}_\theta$	relative slenderness used for calculating room temperature, corrected according to coefficients $k_{y,\theta}$ and $k_{E,\theta}$ obtained in 45.1 with temperature ($\theta_{a,t}$) at the time (t) of the fire process in question;	46.3
$\bar{\lambda}_{c0}$	limit slenderness of the equivalent compressed flange;	35.2.3

$\bar{\lambda}_f$	non-dimensional slenderness of the equivalent compressed flange between bracing points;	35.2.3
$\bar{\lambda}_{LT}$	non-dimensional slenderness for torsional or flexural-torsional buckling;	35.2.2, 35.2.3, 35.2.2.1, 35.3, 46.5
$\bar{\lambda}_{LT,\theta,com}$	non-dimensional slenderness used in room temperature calculations, corrected according to coefficients $k_{y,\theta,com}$ and $k_{E,\theta,com}$;	46.5
$\bar{\lambda}_{LT,0}$	coefficient for calculating lateral buckling in rolled sections or equivalent welded sections subject to bending;	35.2.2.1, 35.2.3
$\bar{\lambda}_0$	non-slenderness of lateral buckling where there is a uniform moment;	35.3
$\bar{\lambda}_p$	non-dimensional slenderness of sheet or panel;	19.5.1, 62.1.2, 73.9.2, 73.11.2, A6.3, A6.4.2
$\bar{\lambda}_{p,red}$	relative slenderness of the sheet or panel in the effective section;	20.7, 73.9.2, 73.9.3, A6.3
$\bar{\lambda}_T$	relative slenderness for torsional or flexural-torsional buckling;	35.1.4
$\bar{\lambda}_w$	relative web slenderness;	35.5.2.1, 35.9.3.3, 73.10
$\bar{\lambda}_{p,ser}$	relative slenderness of the sheet or panel in the effective section for the serviceability limit state;	73.9.2, 73.9.3
μ	ductility performance coefficient. Slip factor. Reduction factor. Efficiency factor;	50.2, 64.6.3, 64.7.3, 66.2, 71.2.3.2, 76.8, A-9-6, A-9-16
ν	Poisson ratio;	20.7, 32.4, 35.5.2.1, 73.9.2, A6.4.2.1, A6.4.2.2, A6.4.3
ξ	factor;	A6.4.4
ρ	width reduction factor of compresion panels. Factor. Specific mass of air. Performance. Buckling reduction factor of equivalent orthotropic plate. Density;	19.5.1, 20.7, 34.7.1, 34.7.3, 62.1.2, 73.9.1, 73.9.2, A6.4.4, A8.5, A8.6.1
ρ_a	steel density. Factor;	48.1, 48.2, 73.9.3
ρ_c	panel instability reduction coefficient;	A6.3, A6.4.1, A6.4.4

ρ_{loc}	reduction coefficient of each panel calculated according to A6.3 to take into account local buckling;	A6.4.1
ρ_p	density of protection material;	45.2
ρ_{pd}	design value of protection material density;	48.2
ρ_{pk}	characteristic value of protection material density;	48.3
σ	Design compressive strength. Admissible compression stress of neoprene support. Stefan-Boltzmann constant;	32.2, 45.1, 66.1, A8.2.2, A8.6.2
$\sigma_{//}$	direct stresses acting on a normal plane to the axis of the chord;	59.8, 59.11
$\sigma_{\perp}$	direct stresses acting on the throat plane of the chord;	59.8, 59.8.2
σ_1	maximum stress at one panel edge;	A6.4.2.1
σ_2	minimum stress at the opposite edge;	A6.4.2.1
σ_a	average flange stress, calculated with the gross section;	73.7
$\sigma_{c,Ed}$	maximum design compression stress;	20.3, 20.7, A6.3, A6.4.2.2
σ_{co}	direct comparative stress;	41.2
$\sigma_{co,Ed,ser}$	direct comparative stress on the panel for the combination of actions;	41.2
$\sigma_{com,Ed}$	maximum design compressive stress on plane elements;	73.9.1, 73.11.1
$\sigma_{com,Ed,ser}$	maximum design compressive stress on plane members for the serviceability limit state;	73.9.2
σ_{cr}	direct critical local buckling stress of the sheet or panel. Critical elastic stress through buckling due to stiffener torsion;	20.7, 73.9.2, A7.2.1
$\sigma_{cr,c}$	critical elastic column buckling stress for a non-stiffened sheet;	A6.4.3, A6.4.4, A7.2.1
$\sigma_{cr,i}$	ideal critical direct panel local buckling stress, assuming its edges are hinged;	40.2
$\sigma_{cr,p}$	critical local buckling stress of the equivalent sheet in longitudinally stiffened plane members;	A6.4.2, A6.4.2.2, A6.4.4, A7.2.1
$\sigma_{cr,sl}$	critical elastic buckling stress of the equivalent column;	A6.4.2.2, A6.4.3

$\sigma_{Dw}(z,s)$	overstress due to distortion of a symmetrical single-cell box beam subject to outer torsional distribution m(z) along its length;	A3.3.1
σ_{E}	Euler critical stress;	35.5.2.1, 40.2
$\sigma_{Ed,ser}$	maximum compression on the panel for the combination of actions;	41.2
$\sigma_{H,ser}$	local contact pressure between pin and piece at serviceability limit state, where the pin must be removable;	58.9
$\sigma_{m\alpha x}$	maximum direct stress;	21.3.5
σ_{mvn}	minimum direct stress;	21.3.5
$\sigma_{My,Ed}$	direct stress to due to bending moment $M_{y,Ed}$, using the effective section;	73.11.1
$\sigma_{Mz,Ed}$	direct stress to due to bending moment $M_{z,Ed}$, using the effective section;	73.11.1
$\sigma_{n,Ed}$	maximum compression stress in the support web;	62.1.2,
$\sigma_{N,Ed}$	axial direct stress, using the effective section;	73.11.1
$\sigma_{o,Ed}$	maximum compression stress on the chord at the joint;	64.2
$\sigma_{p,Ed}$	value of $\sigma_{o,Ed}$ discounting stress due to members parallel to the axis of the chord from axial forces acting on the filler bars at that joint;	64.2
σ_{ref}	reference steel stress level;	32.3
$\sigma_{tot,Ed}$	sum of direct stresses;	73.11.1
$\sigma_{w,Ed}$	direct longitudinal stresses due to warping torsion bi-moment B_{Ed};	34.6
$\sigma_{w,Ed}$	direct stress due to warping torsion, using the gross section;	73.11.1
σ_{x}	direct stress at the point in question;	21.3.5
$\sigma_{x,Ed}$	design value of direct stress in the longitudinal direction at the point in question;	34.1.1, 34.7.2.2, 34.7.2.3, A7.2.4
$\sigma_{x,Ed,ser}$	maximum compression on the panel for the frequent combination of actions;	40.2
$\sigma_{z,Ed}$	design value of direct stress in the transverse direction at the point in question;	21.6, 34.1.1

τ	shear stress;	42.6
$\tau_{//}$	shear stresses acting on the throat plane running parallel to the plane of the chord;	59.8, 59.8.2
$\tau_{\perp}$	shear stresses acting on the throat plane running perpendicular to the plane of the chord;	59.8, 59.8.2
τ_{cr}	shear critical local buckling stress;	35.5.2.1, 73.10
$\tau_{cr,i}$	ideal shear critical panel local buckling stress, assuming its edges are articulated;	40.2
τ_{Ed}	shear stress design value at the point in question;	34.1.1, 34.5
$\tau_{Ed,ser}$	shear stress on the panel for the combination of actions;	40.2, 41.2
$\tau_{t,Ed}$	shear stresses due to torsional force $T_{t,Ed}$ with uniform torsion;	34.6
$\tau_{t,Ed}$	shear stress due to uniform torsion, with the gross section;	73.11.1
$\tau_{tot,Ed}$	sum of shear stresses;	73.11.1
$\tau_{Vy,Ed}$	shear stress to due to shear $V_{y,Ed}$, using the gross section;	73.11.1
$\tau_{Vz,Ed}$	shear stress to due to shear $V_{z,Ed}$, using the gross section;	73.11.1
τ_{w}	average shear stress on the chord;	59.8.2, 59.10, 60.2, 60.2.1
$\tau_{w,Ed}$	shear stress due to warping torsion, with the gross section;	73.11.1
$\tau_{w,Ed}$	shear stresses due to torsional force $T_{w,Ed}$ with warping torsion;	34.6
$\tau_{w,max}$	maximum shear stress on the chord;	60.2.1
φ	list of total coating and steel member heat capacities;	48.2, 48.4
φ	linear defect of verticality. Angle;	22.3.1, 22.3.3, 73.5, 73.6, 73.10, A-9-6, A-9-11, A-9-13, A-9-16
φ_0	lateral imperfection base value;	22.3.1
χ	curvature in diagram M- χ. Reduction coefficient for the form of buckling in question. Reduction coefficient for the equivalent compressed flange	19.5.1, 22.3.5, 35.1.1, 35.1.2, 35.2.3, 35.3,

	determined with $_f\lambda$; ;	35.5.2, Annex 4, A-9-8
χ_c	reduction coefficient for consideration of column-type buckling;	A6.3, A6.4.1, A6.4.3, A6.4.4
χ_{el}	curvature corresponding to the yield strength;	19.5.1
χ_F	local buckling reduction coefficient with a concentrated load;	35.6, 35.7.2
χ_{LT}	reduction coefficient for lateral buckling;	35.2.1, 35.2.2, 35.2.2.1, 35.3
$\chi_{LT,fi}$	reduction coefficient for lateral buckling in a design fire situation;	46.5
$\chi_{LT,mod}$	reduction coefficient modified for lateral-torsional buckling;	35.2.2.1
χ_u	ultimate elastic-plastic curvature;	19.5.1
χ_w	factor for contribution of the web in the case of shear local buckling;	35.5.2, 35.5.2.1, 35.7.1, 35.9.3.3, 35.9.3.5
χ_y	buckling reduction coefficient through bending around the y-y axis;	35.3
χ_z	buckling reduction coefficient through bending around the z-z axis;	35.3
ω	standardised sectorial coordinate;	34.6

A1.2 Units and framework of signs

The units used in this Code correspond to those of the International System of Units, or S.I.

Annex 2: List of UNE standards

The sections of this Code lay down a number of checks concerning the compliance of the products it describes, which, in many cases, refer to UNE, UNE-EN, or UNE-EN ISO standards.

A list of versions of the standards applicable in each case, with reference to their date of approval, is given below.

A2.1 UNE standards

UNE 7475-1:1992	Metallic materials. Charpy impact test. Part 1: Test method.
UNE 14618:2000	Weld inspectors. Qualification and certification.
UNE 36521:1996	Steel products. Hot rolled taper flange I sections (old IPN). Dimensions.
UNE 36522:2001	Steel products. Section UPN. Dimensions.
UNE 36524:1994	Hot rolled steel sections. Broad, parallel faced flanged beams. Dimensions.
UNE 36524:1994 ERRATUM:1999	Hot rolled steel sections. Broad, parallel faced flanged beams. Dimensions.
UNE 36525:2001	Steel products. Section U. Dimensions.
UNE 36526:1994	Hot rolled steel products. IPE beams. Dimensions.
UNE 36559:1992	Hot rolled steel plates 3mm thick or above. Tolerances on dimensions, shape and mass.
UNE 48103:2002	Paints and varnishes. Standardised colours.

A2.2 UNE-EN standards

UNE-EN 287-1:2004	Qualification of test welders. Fusion welding. Part 1: Steels.
UNE-EN 287-1:2004/A2:2006	Qualification of test welders. Fusion welding. Part 1: Steels.
UNE-EN 970:1997	Non-destructive examination of fusion welds. Visual examination.
UNE-EN 1289:1998	Non-destructive examination of welds. Penetrant testing of welds. Acceptance levels.
UNE-EN 1289:1998/1M:2002	Non-destructive examination of welds. Penetrant testing of welds. Acceptance levels.
UNE-EN 1289:1998/A2:2006	Non-destructive examination of welds. Penetrant testing of welds. Acceptance levels.
UNE-EN 1290:1998	Non-destructive examination of welds. Magnetic particle examination of welds.
UNE-EN 1290:1998/1M:2002	Non-destructive examination of welds. Magnetic particle examination of welds.
UNE-EN 1290:1998/A2:2006	Non-destructive examination of welds. Magnetic particle examination of welds.
UNE-EN 1363-1:2000	Fire resistance tests. Part 1: General requirements.

UNE-EN 1363-2:2000	Fire resistance tests. Part 2: Alternative and additional procedures.
UNE-EN 1714	Non-destructive examination of welds. Ultrasonic examination of welds.
UNE-EN 1714:1998/1M:2002	Non-destructive examination of welds. Ultrasonic examination of welds.
UNE-EN 1714:1998/A2:2006	Non-destructive examination of welds. Ultrasonic examination of welds.
UNE-EN 1990:2003	Eurocode. Basis of structural design.
UNE-EN 10024:1995	Hot rolled steel products. Taper flange I sections. Tolerances on dimensions and shape.
UNE-EN 10025-1:2006	Hot rolled products of structural steels. Part 1: Technical delivery conditions.
UNE-EN 10025-2:2006	Hot rolled products of structural steels. Part 2: Technical delivery conditions for non-alloy structural steels.
UNE-EN 10025-3:2006	Hot rolled products of structural steels. Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels.
UNE-EN 10025-4:2007	Hot rolled products of structural steels. Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.
UNE-EN 10025-5:2007	Hot rolled products of structural steels. Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance.
UNE-EN 10025-6:2007 +A1:2009	Hot rolled products of structural steels. Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.
UNE-EN 10034:1994	Structural steel I and H Sections. Tolerances on shape and dimensions.
UNE-EN 10051:1998	Continuously hot-rolled uncoated plate, sheet and strip of non-alloy and alloy steels. Tolerances on dimensions and shape.
UNE-EN 10055:1996	Hot rolled steel equal flange tees with radiused root and toes. Dimensions and tolerances on shape and dimensions.
UNE-EN 10056-1:1999	Specification for structural steel equal and unequal angles. Part 1: Dimensions.
UNE-EN 10056-2:1994	Specification for structural steel equal and unequal angles. Part 2: Tolerances on shape and dimensions.
UNE-EN 10058:2004	Hot rolled flat steel bars for general purposes. Dimensions and tolerances on shape and dimensions.
UNE-EN 10059:2004	Hot rolled square steel bars for general purposes. Dimensions and tolerances on shape and dimensions.
UNE-EN 10060:2004	Hot rolled round steel bars for general purposes. Dimensions and tolerances on shape and dimensions.
UNE-EN 10061:2005	Hot rolled hexagonal steel bars for general purposes. Dimensions and tolerances on shape and dimensions.
UNE-EN 10079:2008	Definition of steel products.
UNE-EN 10083-1:2008	Steels for quenching and tempering. Part 1: General technical delivery conditions.
UNE-EN 10088-1:2006	Stainless steels. Part 1: List of stainless steels.
UNE-EN 10131:2007	Cold rolled uncoated and zinc or zinc-nickel electrolytically coated low carbon and high yield strength

	steel flat products for cold forming. Tolerances on dimensions and shape.
UNE-EN 10149-2:1996	Hot-rolled flat products made of high yield strength steels for cold forming. Part 2: Delivery conditions for thermomechanically rolled steels.
UNE-EN 10149-3:1996	Hot-rolled flat products made of high yield strength steels for cold forming. Part 3: Delivery conditions for normalized or normalized rolled steels.
UNE-EN 10149-3:1996 ERRATUM:2000	Hot-rolled flat products made of high yield strength steels for cold forming. Part 3: Delivery conditions for normalized or normalized rolled steels.
UNE-EN 10162:2005	Cold rolled steel sections. Technical delivery conditions. Dimensional and cross-sectional tolerances.
UNE-EN 10164:2007	Steel products with improved deformation properties perpendicular to the surface of the product. Technical delivery conditions.
UNE-EN 10210-1:2007	Hot finished structural hollow sections of non-alloy and fine grain steels. Part 1: Technical delivery conditions.
UNE-EN 10210-2:2007	Hot finished structural hollow sections of non-alloy and fine grain steels. Part 2: Tolerances, dimensions and sectional properties.
UNE-EN 10219-1: 2007	Cold formed structural hollow sections of non-alloy and fine grain steels. Part 1: Technical delivery conditions.
UNE-EN 10219-2: 2007	Cold formed structural hollow sections of non-alloy and fine grain steels. Part 2: Tolerances, dimensions and sectional properties.
UNE-EN 10268:2007	Cold rolled steel flat products with high yield strength for cold forming. Technical delivery conditions.
UNE-EN 10279:2001	Hot rolled steel channels. Tolerances on shape, dimensions and mass.
UNE-EN 10346:2010	Continuously hot-dip coated flat steel products. Technical delivery conditions.
UNE-EN 12062:1997	Non-destructive testing of welds. General rules for metallic materials.
UNE-EN 12062:1997/ 1M:2002	Non-destructive testing of welds. General rules for metallic materials.
UNE-EN 12517-1:2006	Non-destructive testing of welds. Part 1: Evaluation of welded joints in steel, nickel, titanium and their alloys by radiography. Acceptance levels.
UNE-EN 12517-2:2010	Non-destructive testing of welds. Part 2: Evaluation of welded joints in aluminium and its alloys by radiography. Acceptance levels.
UNE-EN 13438:2007	Paints and varnishes. Powder organic coatings for galvanized or sherardised steel products for construction.
UNE-EN 14399-1:2009	High-strength structural bolting assemblies for preloading. Part 1: General requirements.
UNE-EN 14399-2:2009	High-strength structural bolting assemblies for preloading. Part 2: Suitability test for preloading.
UNE-EN 14399-5:2009	High-strength structural bolting assemblies for preloading. Part 5: Plain washers.
UNE-EN 14399-6:2009	High-strength structural bolting assemblies for preloading. Part 6: Plain chamfered washers.

UNE-EN 15048-1:2008	Non-preloaded structural bolting assemblies. Part 1: General requirements.
UNE-EN 15048-2:2008	Non-preloaded structural bolting assemblies. Part 2: Suitability test.
UNE-EN 15773:2010	Industrial application of powder organic coatings to hot dip galvanized and sherardized steel articles [duplex systems] Specifications, recommendations and guidelines.
UNE-EN 20286-2:1996	ISO system of limits and fits. Part 2: Tables of standard tolerance grades and limit deviations for holes and shafts (ISO 286-2:1988).
UNE-EN 45011:1998	General requirements for bodies operating product certification systems (ISO/IEC guide 65:1996).

A2.3 UNE-EN ISO standards

UNE-EN ISO 643:2004	Steels. Micrographic determination of the apparent grain size (ISO 643:2003).
UNE-EN ISO 1461:2010	Hot dip galvanized coatings on fabricated iron and steel articles. Specifications and test methods. (ISO 1461:2009).
UNE-EN ISO 1716:2002	Reaction to fire tests for building products. Determination of the heat of combustion. (ISO 1716:2002).
UNE-EN ISO 2063:2005	Thermal spraying. Metallic and other inorganic coatings. Zinc, aluminium and their alloys. (ISO 2063:2005).
UNE-EN ISO 2409:2007	Paints and varnishes. Cross-cut test. (ISO 2409:2007).
UNE-EN 2812-1:2007	Paints and varnishes. Determination of resistance to liquids. Part 1: Immersion in liquids other than water. (ISO 2812-1:2007).
UNE-EN ISO 2812-2:2007	Paints and varnishes. Determination of resistance to liquids. Part 2: Water immersion method. (ISO 2812-2:2007).
UNE-EN ISO 3834-1:2006	Quality requirements for fusion welding of metallic materials. Part 1: Criteria for the selection of the appropriate level of quality requirements. (ISO 3834-1:2005).
UNE-EN ISO 4014:2001	Hexagon head bolts. Product grades A and B (ISO 4014:1999).
UNE-EN ISO 4016:2001	Hexagon head bolts. Product grade C (ISO 4016:1999).
UNE-EN ISO 4017:2001	Hexagon head screws. Product grades A and B (ISO 4017:1999).
UNE-EN ISO 4018:2001	Hexagon head screws. Product grade C (ISO 4018:1999).
UNE-EN ISO 4032:2001	Hexagon nuts. Type 1. Product grades A and B (ISO 4032:1999).
UNE-EN ISO 4033:2001	Hexagon nuts. Type 2. Product grades A and B (ISO 4033:1999).
UNE-EN ISO 4034:2001	Hexagon nuts. Product grade C (ISO 4034:1999).
UNE-EN ISO 4063:2010	Welding and allied processes. Nomenclature of processes and reference numbers. (ISO 4063:2009).
UNE-EN ISO 4624:2003	Paints and varnishes. Pull-off test for adhesion. (ISO 4624:2002).
UNE-EN ISO 4628-2:2004	Paints and varnishes. Evaluation of degradation of coatings. Designation of quantity and size of defects, and

	of intensity of uniform changes in appearance. Part 2: Assessment of degree of blistering (ISO 4628-2:2003).
UNE-EN ISO 4628-3:2004	Paints and varnishes. Evaluation of degradation of coatings. Designation of quantity and size of defects, and of intensity of uniform changes in appearance. Part 3: Assessment of degree of rusting (ISO 4628-3:2003).
UNE-EN ISO 4628-4:2004	Paints and varnishes. Evaluation of degradation of coatings. Designation of quantity and size of defects, and of intensity of uniform changes in appearance. Part 4: Assessment of degree of cracking (ISO 4628-4:2003).
UNE-EN ISO 4628-5:2004	Paints and varnishes. Evaluation of degradation of coatings. Designation of quantity and size of defects, and of intensity of uniform changes in appearance. Part 5: Assessment of degree of flaking (ISO 4628-5:2003).
UNE-EN ISO 5817:2009	Welding. Fusion-welded joints in steel, nickel, titanium and their alloys (beam welding excluded). Quality levels for imperfections (ISO 5817:2003, corrected version: 2005, including Technical Corrigendum 1:2006).
UNE-EN ISO 6270-1:2002	Paints and varnishes. Determination of resistance to humidity. Part 1: Continuous condensation (ISO 6270:1998).
UNE-EN ISO 6507-1:2006	Metallic materials. Vickers hardness test. Part 1: Test method (ISO 6507-1:2005).
UNE-EN ISO 6520-1:2009	Welding and allied processes. Classification of geometric imperfections in metallic materials. Part 1: Fusion welding (ISO 6520-1:2007).
UNE-EN ISO 6892-1:2010	Metallic materials. Tensile testing. Part 1: Method of test at room temperature. (ISO 6892-1:2009)
UNE-EN ISO 7089:2000	Plain washers. Normal series. Product grade A (ISO 7089:2000).
UNE-EN ISO 7090:2000	Plain chamfered washers. Normal series. Product grade A (ISO 7090:2000).
UNE-EN ISO 7091:2000	Plain washers. Normal series. Product grade C (ISO 7091:2000).
UNE-EN ISO 7092:2000	Plain washers. Small series. Product grade A (ISO 7092:2000).
UNE-EN ISO 7093-1:2000	Plain washers. Large series. Part 1. Product grade A (ISO 7093-1:2000).
UNE-EN ISO 7093-2:2000	Plain washers. Large series. Part 2. Product grade C (ISO 7093-2:2000).
UNE-EN ISO 7094:2000	Plain washers. Extra large series. Product grade C (ISO 7094:2000).
UNE-EN ISO 7438:2006	Metallic materials. Bend test. (ISO 7438:2005).
UNE-EN ISO 8501-1:2008	Preparation of steel substrates before application of paints and related products. Visual assessment of surface cleanliness. Part 1: Rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings (ISO 8501-1:2007).
UNE-EN ISO 8502-3:2000	Preparation of steel substrates before application of paints and related products. Tests for the assessment of surface cleanliness. Part 3. Assessment of dust on steel surfaces prepared for painting (pressure-sensitive tape method) (ISO 8502-3:1992).

UNE-EN ISO 8503-1:1996	Preparation of steel substrates before application of paints and related products. Surface roughness characteristics of blast-cleaned steel substrates. Part 1: Specifications and definitions for ISO surface profile comparators for the assessment of abrasive blast-cleaned surfaces (ISO 8503-1:1988).
UNE-EN ISO 8503-2:1996	Preparation of steel substrates before application of paints and related products. Surface roughness characteristics of blast-cleaned steel substrates. Part 2: Method for the grading of surface profile of abrasive blast-cleaned steel. Comparators procedure (ISO 8503-1:1988).
UNE-EN ISO 8503-3:1996	Preparation of steel substrates before application of paints and related products. Surface roughness characteristics of blast-cleaned steel substrates. Part 3. Method for the calibration of ISO surface profile comparators and for the determination of surface profile. Focusing microscope procedure (ISO 8503-3:1988).
UNE-EN ISO 8503-4:1996	Preparation of steel substrates before application of paints and related products. Surface roughness characteristics of blast-cleaned steel substrates. Part 4. Method for the calibration of ISO surface profile comparators and for the determination of surface profile. Stylus instrument procedure (ISO 8503-4:1988).
UNE-EN ISO 8503-5:2006	Preparation of steel substrates before application of paints and related products. Surface roughness characteristics of blast-cleaned steel substrates. Part 5: Replica tape method for the determination of the surface profile (ISO 8503-5:2003).
UNE-EN ISO 8504-2:2002	Preparation of steel substrates before application of paints and related products. Surface preparation methods. Part 2: Abrasive blast-cleaning (ISO 8504-2:2000).
UNE-EN ISO 8504-3:2002	Preparation of steel substrates before application of paints and related products. Surface preparation methods. Part 3: Hand- and power-tool cleaning (ISO 8504-3:1993).
UNE-EN ISO 8504-1:2002	Preparation of steel substrates before application of paints and related products. Surface preparation methods. Part 1: General principles (ISO 8504-1:2000).
UNE-EN ISO 9001:2008	Quality management systems. Requirements (ISO 9001:2008).
UNE-EN ISO 9227:2007	Corrosion tests in artificial atmospheres. Salt spray test (ISO 9227:2006).
UNE-EN ISO 9692-1:2004	Welding and allied processes. Recommendations for joint preparation. Part 1: Manual metal-arc welding, gas-shielded metal-arc welding, gas welding, TIG welding and beam welding of steels (ISO 9692-1:2003).
UNE-EN ISO 10666:2000	Drilling screws with tapping screw thread. Mechanical and functional properties (ISO 10666:1999).
UNE-EN ISO 10684:2006	Fasteners. Hot-dip galvanized coatings (ISO 10684:2004).

UNE-EN ISO 10684:2006/ AC:2009	Fasteners. Hot-dip galvanized coatings (ISO 10684:2004/Cor 1:2008).
UNE-EN ISO 12944-1:1999	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 1: General introduction (ISO 12944-1:1998).
UNE-EN ISO 12944-2:1999	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 2: Classification of environments. (ISO 12944-2:1998).
UNE-EN ISO 12944-3:1999	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 3: Design considerations. (ISO 12944-3:1998).
UNE-EN ISO 12944-4:1999	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 4: Types and surface preparation (ISO 12944--4:1998).
UNE-EN ISO 12944-5:2008	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 5: Protective paint systems. (ISO 12944-5:2008).
UNE-EN ISO 12944-6:1999	Paints and varnishes. Corrosion protection of steel structures by protective paint systems. Part 6: Laboratory performance test methods. (ISO 12944-6:1998).
UNE-EN ISO 13918:2009	Welding. Studs and ceramic ferrules for arc stud welding (ISO 13918:2008).
UNE-EN ISO 13920:1997	Welding. General tolerances for welded constructions. Dimensions for lengths and angles. Shape and position. (ISO 13920:1996).
UNE-EN ISO 14713:2000	Protection against corrosion of iron and steel in structures. Zinc and aluminium coatings. Guidelines (ISO 14713:1999).
UNE-EN ISO 14731:2008	Welding coordination. Tasks and responsibilities. (ISO 14731:2006)
UNE-EN ISO 15480:2000	Hexagon washer head drilling screws with tapping screw thread. (ISO 15480:1999).
UNE-EN ISO 15481:2000	Cross recessed pan head drilling screws with tapping screw thread. (ISO 15481:1999).
UNE-EN ISO 15482:2000	Cross recessed countersunk head drilling screws with tapping screw thread. (ISO 15482:1999).
UNE-EN ISO 15483:2000	Cross recessed raised countersunk head drilling screws with tapping screw thread. (ISO 15483:1999).
UNE-EN ISO 15607:2004	Specification and qualification of welding procedures for metallic materials. General rules (ISO 15607:2003).
UNE-EN ISO 15609-1:2005	Specification and qualification of welding procedures for metallic materials. Welding procedure specification. Part 1: Arc welding. (ISO 15609-1:2004).
UNE-EN ISO 15613:2005	Specification and qualification of welding procedures for metallic materials. Qualification based on pre-production welding test (ISO 15613:2004).
UNE-EN ISO 15614-1:2005	Specification and qualification of welding procedures for metallic materials. Welding procedure test. Part 1: Arc and gas welding of steels and arc welding of nickel and nickel alloys (ISO15614-1:2004).
UNE-EN ISO 15792-1	Welding consumables. Test methods. Part 1: Test methods for all-weld metal test specimens in steel, nickel and nickel alloys (ISO 15792-1:2000).

A2.4 UNE-EN ISO/IEC standards

UNE-EN ISO/IEC 17021:2006	Conformity assessment. Requirements for bodies providing audit and certification of management systems (ISO/IEC 17021:2006)
UNE-EN ISO/IEC 17025:2005	Conformity assessment. General requirements for the competence of testing and calibration laboratories.
UNE-EN ISO/IEC 17025:2005 ERRATUM:2006	Conformity assessment. General requirements for the competence of testing and calibration laboratories (ISO/IEC 17025:2005/Cor. 1:2006).

A2.5 Other standards.

EN 1090-2: 2008	Execution of steel structures and aluminium structures. Part 2: Technical requirements for the execution of steel structures.
EN 1990	Eurocode 0: Basis of structural design.
EN 1991	Eurocode 1: Actions on structures.
EN 1993	Eurocode 3: Design of steel structures.
EN 1993-1-3:2006	Eurocode 3: Design of steel structures. Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting.
EN 1993-1-3:2006	Eurocode 3: Design of steel structures. Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting.
EN 1993-1-3:2006/AC:2009	Eurocode 3: Design of steel structures. Part 1-3: General rules. Supplementary rules for cold-formed members and sheeting.
EN 1993-1-6:2007	Eurocode 3: Design of steel structures. Part 1-6: General rules. Strength and stability of shell structures.
EN 1993-1-6:2007/AC:2009	Eurocode 3: Design of steel structures. Part 1-6: General rules. Strength and stability of shell structures.
EN 08.01.93:2005	Eurocode 3: Design of steel structures. Part 1-8: Design of joints.
EN 1993-1-8:2005/AC:2009	Eurocode 3: Design of steel structures. Part 1-8: Design of joints.
EN 1993-2: 2006	Eurocode 3: Design of steel structures. Part 2. Steel bridges.
EN 1994-2: 2005	Eurocode 4: Design of composite steel and concrete structures. Part 1-1. General rules and rules for buildings.
EN 1997	Eurocode 7: Geotechnical design.
EN 1998	Eurocode 8: Design of structures for earthquake resistance.
EN 14399-9: 2009	High-strength structural bolting assemblies for preloading. Part 9: System HR or HV. Direct tension indicators for bolt and nut assemblies.
ISO 7976-1:1989	Tolerances for building -- Methods of measurement of buildings and building products -- Part 1: Methods and instruments.
ISO 7976-2:1989	Tolerances for building -- Methods of measurement of buildings and building products -- Part 2: Methods and instruments.

ISO 9226:1992	Corrosion of metals and alloys. Corrosivity of atmospheres. Determination of corrosion rate of standard specimens for the evaluation of corrosivity.
ISO17123	Optics and optical instruments. Field procedures for testing geodetic and surveying instruments.
UNE-ENV 1090-2:1999	Execution of steel structures. Part 2: Supplementary rules for cold-formed members and sheeting.
UNE-ENV 13381-2:2004	Test methods for determining the contribution to the fire resistance of structural members. Part 2: Vertical protective membranes.
UNE-ENV 13381-4:2005	Test methods for determining the contribution to the fire resistance of structural members. Part 4: Applied protection to steel members.
CEN/TS 13381-1:2005	Test methods for determining the contribution to the fire resistance of structural members. Part 1: Horizontal protective membranes.

Annex 3: Diaphragms

A3.1 General information and scope of application

The box girder analysis may be performed using non-warping beam models, unaffected by cross section distortion, provided that in addition to the stiff diaphragms located in support sections, intermediate transverse diaphragms are also spaced modularly along the beam, which limit the effects of box distortion and are thus of great importance to the project.

This Annex lays down conditions for governing the design of such intermediate diaphragms used in road bridges, with a cross section formed by a symmetrical vertical axis and which comprise an open, single-cell steel box section that is closed off above by a concrete slab.

This Annex also proposes methods for quantifying the impact of cross section distortion on the response of a symmetrical single-cell box girder with a given configuration of diaphragms along its length. The method is generally applied to structures subject to significant torsional actions and where it is necessary to identify the effects of distortion in their design, wherever the established geometric limitations may apply. The advantage of the proposed method is that it allows for a maintained non-warping girder section methodology during analysis, which is the most common practice in box girder designs, with the subsequent addition, as a further stress and strain condition, of the impact of distortion on each of the load possibilities considered.

A3.2 Measuring of diaphragms in road bridges

A3.2.1 Minimum geometric conditions

The separation between consecutive diaphragms shall not exceed for times the depth of the metal section, and there shall be at least four intermediate diaphragms per span, in addition to the stiff diaphragms located in support sections.

The provisions of this Annex are applicable to plate diaphragms or any of the lattice-type diaphragms indicated in section A3.3, provided that, in the latter case, the diagonals are not tapered with the lower horizontal at 2.5H/1V. Plate diaphragms may contain recesses giving access to the box interior, provided that their dimensions do not significantly compromise the stiffness on the plane of the diaphragm.

Stiff-frame diaphragms consisting of stiffeners perimeter-welded to the main sheets of the box may also be used. However, such types of diaphragm may be substantially less effective in controlling the effects of box distortion than the two previous groups, therefore the provisions and methods in this Annex apply only to frame-type diaphragms where the frame achieves a stiffness equal to that obtained using a lattice diaphragm measured following the criteria set forth in this Annex.

Diaphragms are to be situated on substantially normal planes of the box directrix. Deviations exceeding 10° on the ground and 5° in elevation between the plane of the diaphragm and the box directrix are outside the scope of this Annex.

A3.2.2 Diaphragm resistance measurement

Diaphragms are generally measured following criteria of resistance to the loads to which they are subject, of which consideration must be given to those originating from:

a) Cross section development of uniform torsion flows, which balance out exterior torsional actions affecting the box, and, for, curved bridges, torsion due to deviation in plan of concentrated deck bending.
b) Loads acting directly upon any element of the diaphragm, where the diaphragms form part of the platform-derived load transmission system.
c) Resistance to actions transmitted by any lateral cantilever systems that may exist in wide bridges.
d) Deviation forces created by curvature in elevation of the box, or localised or gradual changes in inclination of the lower flange in variable-depth bridges.
e) Transfer of localised forces generated in individual areas of the deck with a certain geometric discontinuity, sharp cross sectional changes, or any other circumstance that implies a dramatic and localised alteration of the tensile properties of the box.
f) In trapezoidal boxes, absorption of the horizontal component, allowing for the balancing of stresses deriving from exterior loads applied to the platform, which are generally vertical and with shear tensile flows on the inclined planes of the webs.
g) Forces generated during structure assembly that can have an effect on the open box section, before it is closed off with the upper compound section slab, and for which the design must include a suitable resistance mechanism. Particular consideration must be given to:

    – Any torsional stresses that may be introduced before closure of the section, either permanent or temporary, during concreting of the upper slab.
    – For trapezoidal boxes, impact of the inclination of webs on the balance between exterior loads applied and shear flows from the section that balances them.
    – Lateral buckling restraint of compressed chords under lateral buckling during assembly.
    – Action of the wind on the open section.

h) All stresses deriving from functions typical of conventional transverse stiffeners in webs and flanges, when, as is often the case, these simultaneously carry out their own stiffening functions on diaphragms, namely:

    – Controlling local buckling of web panels and flanges subject to direct and shear stresses.
    – Buckling restraint of compressed web and flange stiffeners in compressed areas.
    – Gathering deviation forces due to the possible curvature or alignment changes of web or flange sheets and their stiffeners.
    – Absorption of actions applied directly to the faces of the box, either deriving from the platform and acting upon the upper member of the diaphragm, or applied directly to the sheeting panels, primarily in the form of wind.

For each of the stresses described, depending on the type of the diaphragm, a test model will be drawn up in order to determine the forces acting upon the different elements of the diaphragm and for its subsequent measurement, according to the applicable rules described in the articles of this Code.

As a general rule, the model for determining forces on the diaphragm includes an ideal box section sample, with cross-sections before and after the diaphragm on its rear and front sections, respectively. These are subjected to actions resulting from the stresses described above, which act upon the diaphragm, along with the action from front and rear shear flows acting on the webs and flanges of the box which balances the stresses, such that the system of actions introduced remains continuously self-balanced.

Where the stiffness of a diaphragm has a significant impact on the strains upon it, the distribution of localised actions between consecutive diaphragms may be considered to determine the diaphragm's dimensional stresses, providing such distribution is justified in the project by the corresponding model. However, unless this justification is included in the design, a diaphragm must be able to fully withstand all actions applied directly to it, without the support of any of the adjacent diaphragms. As an exception, an isostatic distribution between adjacent diaphragms of the effects of actions localised between them is allowed without further justification.

Sections A3.3 and A3.4 include a test method for considering within the design the distribution between diaphragms of stresses resulting from the introduction of uniform torsional flows from localised actions into the box section, providing the relevant geometric limitations are satisfied. For actions of other origins, the Designer should justify any distribution between different diaphragms with the corresponding model, according to the nature of the issue to be resolved and the warping properties of both box and diaphragms.

Once the stresses on the different elements of the diaphragm are obtained, they can be measured in each case according to the articles of this Code. For linear members obtained by welding a stiffener to a panel of web or flange sheeting, strength tests will consider collaboration with the stiffener of a portion of sheet with a width of 15 ε t on each side, where t is the thickness of the web or flange sheet and $\varepsilon = \sqrt{(235/f_y)}$, with f_y in N/mm^2.

Where the verification in ultimate limit states considers plastic strength, in the case of grade 1 or 2 sections, the test will also check for the absence of local plastifications at serviceability limit state.

Moreover, two-dimensional elements (plate diaphragms) shall also be subject to buckling tests under the service loads set out in this Code.

A3.2.3 Required diaphragm stiffness

In addition to the strength tests described, the dimensions of diaphragms must also provide the necessary stiffness to withstand forces resulting from the stresses indicated above, without warping, which could significantly alter the general response of the bridge established in the general dimensional design or hypothesis, which have served as a foundation for the design of the diaphragm.

Provided that diaphragms are suitably sized for resistance conditions in compliance with the previous section, fulfilment of the minimum geometric conditions established in the articles of this Code for each of the elements which make up the diaphragm (maximum thicknesses in linear and plain elements, minimum sheet thicknesses, minimum geometric conditions in lattice and stiffener elements, minimum geometric conditions in joints, etc.) generally provides a guarantee of its stiffness, with the following additional precautions:

a) Where the elements of a diaphragm are also to assume the conventional functions of webs or flanges, they must therefore also satisfy the conditions of stiffness required of such elements in the applicable articles of this Code.

b) Where the warping of a diaphragm or any element thereof could significantly affect an ultimate or serviceability limit state test, consideration must be given to the impact of the diaphragm's flexibility on the test.

c) For straight composite road bridges with a symmetrical single-cell box section, except in relation to the fatigue limit state, the effects of box distortion due to insufficient diaphragm stiffness may be prevented in ULS or SLS tests, providing the following conditions are met:

   – Each cross section web follows a single plain with an inclination of less than 45° against the vertical.
   – The width/light ratio of the box does not exceed 0.40, considering as the width only the maximum separation between webs.
   – The width/depth ratio of the box does not exceed 8, with the same definition of width as above.
   – The conditions of A3.2.1 are satisfied in regard to the number, separation and orientation of diaphragms.
   – Rigid diaphragms are installed in all support sections.
   – Intermediate diaphragms between supports are suitably sized for resistance conditions in compliance with section A3.2.2 above and in the case of rigid diaphragms, i.e. without considering any distribution between adjacent diaphragms of localised actions applied directly to a diaphragm.
   – The box geometry is constant or varies only slightly along the directrix, except in a certain number of localised sections containing rigid diaphragms which are sufficiently separated to allow for the development of direct stresses.

d) Diaphragms sized according to the previous conditions guarantee an overstress of less than 25N/mm^2 in the lower metal corners of the box for the load train established in the Code on actions to be considered in road bridge design IAP-98. In the rest of the cross-section, the stress distribution can be obtained from Figure A3.2. Where the fatigue checking of details corresponding to flanges and webs of the box may be determinant of the design, these overstresses shall be considered in the corresponding fatigue verification, corrected to take account the actual magnitude of the fatigue vehicle which shall be used in that verification, according to regulations, and which is smaller than the load used for resistance verification. Nevertheless it shall not be necessary to consider the overstresses in lower flanges with double mix action neither in compression upper flanges connected to the upper slab.

Where the fatigue verification is a determining factor and the indicated maximum torsional overstress determines the validity of a given detail, it is advisable to modify the details of the design to improve its category, in order to gain a satisfactory degree of fatigue control. However, given that the indicated 25 N/mm^2 of torsional overstress corresponds to the range of boxes with the least favourable geometry from those described in this section, and with the maximum acceptable separation between adjacent diaphragms, a more specific calculation of the torsional overstress may be carried out as indicated in the general method in section A3.3.

A3.3 General calculation of the effects of distortion

A3.3.1 Calculation of distortional overstress

For a more precise determination of the effects of distortion associated with a given diaphragm configuration, the following method allows for the relation of the diaphragm configuration along the box beam, characterised by separation between consecutive diaphragms and their stiffness, with the distortional overstress and warping observed on the box.

The proposed method is generally applicable to all types of box beam, including bridge, road or railway girders or those of any other type, provided that the following geometric conditions are satisfied:

- The box directrix is straight and with no skews in supports.
- The box cross section is a symmetrical single-cell which remains constant along the directrix.
- The depth/light and width/light ratios of the box do not exceed 0.40, considering as the width only the maximum separation between webs.

However, the general application of the method may also be extended to more complex geometries:

- Boxes with curvature on the ground.
- Boxes with a variable geometry, provided that the variation is gradual along the directrix, and there are sharp geometrical changes in localised sections which contain rigid diaphragms.
- Boxes with skews in the supports, which contain rigid diaphragms.

In these cases, the method allows for an approximation of the impact of distortion on general box response, and can serve as a suitable design tool provided that box distortion is only moderate and is not a determining factor in its resistance response, and that localised distortion in individual sites (support zones, skews, changes in geometry, etc.) is controlled by rigid diaphragms. Where distortion does have a significant impact, more developed techniques must be used, with folded plates or finite members, which allow for a more precise evaluation of the longitudinal and transverse performance of the box.

The test method is based on the existing analogy between the distortion of symmetrical single-cell cross section box beams subject to eccentric actions and the bending of linear members over the elastic foundation subject to vertical loads. In compliance with this analogy, overstress due to the distortion of a symmetrical single-cell box beam subject to exterior torsional distribution m(z) along its length is given by the expression:

$$\sigma_{Dw}(z,s) = \frac{B_D(z)\,\omega(s)}{I_b}$$

Where:

I_b is inertia to the distortional warping of the section, which depends solely on the geometry of the cross section and is given by the expression:

$$I_b = \frac{2\frac{a_B}{a_T}\left[(\alpha_T+2)(\alpha_B+2)-1\right]}{(1+\frac{a_B}{a_T})(3+3\frac{a_B}{a_T}+\alpha_T+\alpha_B\frac{a_B}{a_T})}\overline{I}_W$$

with: $\alpha_T = \frac{t_T\,(a_T+2a_c)^3}{t_W d_W a_T^2}$ $\qquad \alpha_B = \frac{t_B a_B}{t_W d_W}$ $\qquad \overline{I}_W = 1/12\; t_W\, d_W^3$

a_T, a_B, a_c, t_T, t_B, t_W, d_W, values which define the geometry of the cross section according to the following figure:

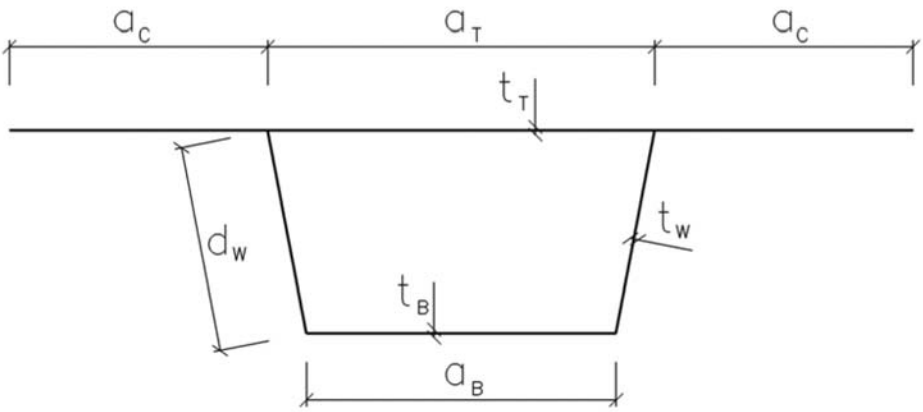


Figure A3.3.1.a

ω(s) is the transverse distribution of direct stresses on the section, and is given by the following figure:

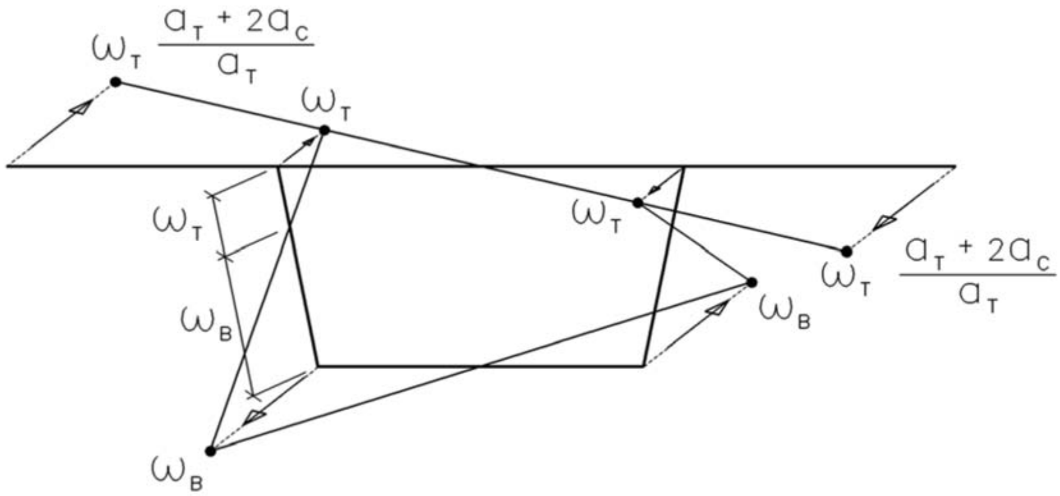


Figure A3.3.1.b

where ω_T and ω_B are given by the following expressions:

$$\omega_T = \frac{1+(2+\alpha_B)\dfrac{a_B}{a_T}}{3+\alpha_T+(3+\alpha_B)\dfrac{a_B}{a_T}}\, d_W \qquad\qquad \omega_B = d_W - \omega_T$$

B_D (z) is the distortional bi-moment on the beam, function of the longitudinal coordinate of beam z, and which coincides with the law of bending moments on an equivalent beam defined as follows:

– The equivalent beam is straight and of the same length as the real beam. Where the method is applied to curved beams, the length of the equivalent beam shall be taken as the developed length of the curved beam.
– The inertia of the beam is given by the inertia to distortional warping I_b defined above. Where the method is applied to boxes with a non-constant geometry, I_b shall also vary along the beam with its real variation.
– The beam is supported with elastic links at the diaphragm installation points. The stiffness constant K_D of the supports depends on the type of diaphragm, and is indicated further on for the most common cases.
– Support sections have rigid links, which must materialise in the real structure with rigid diaphragms.
– A rigid diaphragm must also be used in sections where there is a sharp change in box geometry, when the method is applied as an approximation to such types of box. In the equivalent beam, this diaphragm may be represented with the stiffness constant K_D corresponding to its configuration, as indicated below. Generally, this diaphragm must be configured with a significantly higher stiffness than all other adjacent diaphragms.
– The vertical load p(z) applied to the beam depends on the geometry of the box and the exterior torsional distribution m(z) applied to the box for which the distortion is tested, and is given by the expression:

$$p(z) = m(z)\left(\frac{d_W}{a_T b} - \frac{d_W}{\Omega}\right)$$

where Ω is double the internal area of the trapezoid that forms the cross section, while all the other geometric parameters are defined in the figure above. Where the beam has a curvature on the ground, in addition to the exterior torsions introduced onto the beam m(z), the additional torsion that produces the deviation on the ground of bends in the beam, obtained by dividing the law of bending moments on the beam by the radius of curvature at each point, must also be included.

The stiffness constant K_D in elastic links represents the stiffness of the diaphragm in its plain which counteracts distortional warping, and assumes the following values for the types of diaphragms considered in this Annex:

Plate diaphragm

$$K_D = \frac{1}{4} G t_d f_d^2 \frac{L_p^2}{A_i} \quad \text{con} \quad f_d = \frac{2\left(1+\dfrac{a_T}{a_B}\right)}{\sqrt{1+\left(\dfrac{a_T+a_B}{2b}\right)^2}}$$

where, t_d is the diaphragm sheet thickness, L_p the geometric diagonal of the box cross-section, and A_i the area of the trapezoid that forms the cross section.

St. Andrew's Cross with two composite bars:

$$K_D = \frac{1}{2} E A_d \frac{f_d^2}{L_b} \quad \text{con} \quad f_d = \frac{2\left(1+\dfrac{a_T}{a_B}\right)}{\sqrt{1+\left(\dfrac{a_T+a_B}{2b}\right)^2}}$$

where A_d is the area of the diagonal of the lattice and L_b its length.

Individual diagonal or tensile St. Andrew’s Cross:

$$K_D = \frac{1}{4} E A_d \frac{f_d^2}{L_b} \quad \text{con} \quad f_d = \frac{2\left(1+\dfrac{a_T}{a_B}\right)}{\sqrt{1+\left(\dfrac{a_T+a_B}{2b}\right)^2}}$$

with the same meanings as above.

V lattice:

$$K_D = \frac{1}{4} \frac{E A_d f_d^2}{2L_b} \quad \text{con:} \quad f_d = \frac{2b\left(1+\dfrac{a_T}{a_B}\right)}{L_b}$$

where A_d is the area of the diagonal of the lattice and L_b its length.

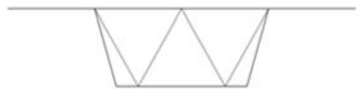
Inverted V lattice:

$$K_D = \frac{1}{4} \frac{E A_d f_d^2}{2L_b} \quad \text{con:} \quad f_d = \frac{2b\left(1+\dfrac{a_T}{a_B}\right)}{L_b}$$

with the same meanings as above.

Double V lattice:

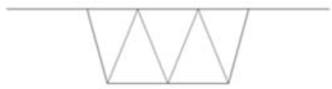
$$K_D = \frac{EA_d}{L_b^3} b^2$$



where A_d is the area of the diagonal of the lattice and L_b its length.

Double inverted V lattice:

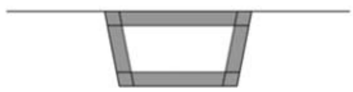
$$K_D = \frac{1}{2}\frac{EA_d\, f_{aux,d}^2}{8L_b} \quad \text{con} \quad f_{aux,d} = \frac{2b\left(1+\dfrac{2a_T - a_B}{a_B}\right)}{L_b}$$



where A_d is the area of the diagonal of the lattice and L_b its length.

Frame diaphragm:

$$K_D = \frac{24\, d_W}{a_B^2\, b^2} C_1 C_2\, EI_W$$



where I_W is the second moment of area of the frame member corresponding to the box web, obtained by adding to the web stiffener a support width on each side of 15 ε t, with ε= √ (275/f_y), f_y en N/mm^2.

The parameters C_1, C_2, r_T , r_B are:

$$C_1 = \frac{(1+\frac{a_B}{a_T})(2+2\frac{a_B}{a_T}+2(\frac{a_B}{a_T})^2+\alpha_T+\alpha_B(\frac{a_B}{a_T})^2)}{3+3\frac{a_B}{a_T}+\alpha_T+\alpha_B\frac{a_B}{a_T}}$$

$$C_2 = \frac{2+2\frac{a_B}{a_T}+2(\frac{a_B}{a_T})^2+r_T+r_B(\frac{a_B}{a_T})^2}{\frac{a_B}{a_T}\left[(r_T+2)(r_B+2)-1\right]}$$

$$r_T = \frac{I_W b_T}{I_T d_W} \qquad\qquad r_B = \frac{I_W b_B}{I_B d_W}$$

The stress thus obtained coincides with the distortional overstress on the box for the load state corresponding to the torsional distribution introduced.

A3.3.2 Calculation of warping due to distortion

The deflection obtained on the equivalent beam coincides with the displacement of the web in its plane due to the distortion of the cross section, which must be considered when testing the serviceability limit states, where relevant.

A3.3.3 Interaction between diaphragm sizing and longitudinal calculation.

The general method laid out in A3.3 also allows for relation between the general response of the box and the sizing of the diaphragms, since the reactions on the elastic springs of the equivalent beam model reflect the distribution between different diaphragms of the localised torsional actions. The geometric limitations for the application of the method described in this section are the same as those illustrated in the previous section.

The real stresses S_D on the different elements of the diaphragm can be estimated based on those obtained in the hypothesis of fixed diaphragms $S_{D,rvgido}$, from section A3.2.2, by means of the expression:

$$S_D = F S_{D,rvgido}$$

in which the factor of proportionality F is given by the quotient:

$$F = \frac{R_D}{R_{D,rígido}}$$

in other words, by the relation between reactions on the springs obtained in the equivalent beam model and those obtained in the same model but with fixed supports in the sections where the diaphragms are located.

The method thus allows for the optimised sizing of diaphragms for stresses generally less intense than those obtained in the case of full stiffness when estimating forces on diaphragms.

However, where diaphragms are sized in consideration of this flexible distribution between consecutive diaphragms of stresses generated by the introduction of localised torsional actions, consideration must also be given to the effects of distortion on tests at serviceability limit state (warping, local plastification and local buckling) and at the ultimate limit state of fatigue, placing overstresses and distortional warping directly over those obtained in the analysis of the box as a one-dimensional beam-type element.

In general, distortional overstress can be ruled out in tests at ultimate limit states, except fatigue, provided that its increased value does not exceed 10 % of the reduced yield strength of the material.

Annex 4: European buckling curves

This Annex gives the “source” values of the buckling reduction coefficient χ in relation to non-dimensional slenderness $\bar{\lambda}$, in the form of tables χ - $\bar{\lambda}$, for the different buckling curves considered.

"a" Curve

$\bar{\lambda}$	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.2	1.0000	0.9983	0.9966	0.9948	0.9930	0.9910	0.9891	0.9872	0.9852	0.9833
0.3	0.9813	0.9794	0.9775	0.9756	0.9737	0.9719	0.9700	0.9682	0.9664	0.9645
0.4	0.9627	0.9608	0.9590	0.9571	0.9552	0.9533	0.9515	0.9496	0.9477	0.9459
0.5	0.9440	0.9421	0.9403	0.9384	0.9366	0.9346	0.9327	0.9308	0.9288	0.9269
0.6	0.9249	0.9229	0.9208	0.9188	0.9168	0.9148	0.9129	0.9108	0.9087	0.9065
0.7	0.9040	0.9013	0.8982	0.8949	0.8914	0.8876	0.8836	0.8794	0.8751	0.8708
0.8	0.8659	0.8610	0.8560	0.8509	0.8456	0.8401	0.8345	0.8267	0.8228	0.8166
0.9	0.8103	0.8039	0.7973	0.7905	0.7838	0.7765	0.7692	0.7618	0.7543	0.7467
1.0	0.7390	0.7313	0.7235	0.7157	0.7078	0.6999	0.6920	0.6840	0.6761	0.6681
1.1	0.6601	0.6522	0.6443	0.6364	0.6286	0.6208	0.6131	0.6055	0.5979	0.5904
1.2	0.5831	0.5758	0.5685	0.5614	0.5543	0.5473	0.5404	0.5336	0.5268	0.5202
1.3	0.5136	0.5071	0.5007	0.4944	0.4882	0.4820	0.4760	0.4701	0.4643	0.4586
1.4	0.4529	0.4474	0.4419	0.4366	0.4313	0.4261	0.4209	0.4159	0.4109	0.4060
1.5	0.4011	0.3964	0.3917	0.3871	0.3828	0.3781	0.3737	0.3694	0.3651	0.3610
1.6	0.3569	0.3528	0.3488	0.3449	0.3410	0.3372	0.3335	0.3298	0.3262	0.3226
1.7	0.3191	0.3156	0.3122	0.3089	0.3056	0.3023	0.2991	0.2959	0.2928	0.2898
1.8	0.2868	0.2838	0.2809	0.2780	0.2752	0.2724	0.2696	0.2669	0.2642	0.2618
1.9	0.2590	0.2564	0.2539	0.2514	0.2489	0.2465	0.2441	0.2418	0.2395	0.2372
2.0	0.2349	0.2327	0.2305	0.2284	0.2262	0.2241	0.2220	0.2200	0.2180	0.2160
2.1	0.2140	0.2121	0.2102	0.2083	0.2064	0.2046	0.2028	0.2010	0.1992	0.1974
2.2	0.1957	0.1940	0.1923	0.1907	0.1891	0.1875	0.1859	0.1843	0.1827	0.1812
2.3	0.1797	0.1782	0.1767	0.1753	0.1738	0.1724	0.1710	0.1696	0.1683	0.1669
2.4	0.1656	0.1642	0.1629	0.1616	0.1603	0.1591	0.1578	0.1566	0.1554	0.1542
2.5	0.1530	0.1518	0.1506	0.1495	0.1483	0.1472	0.1461	0.1450	0.1439	0.1428
2.6	0.1417	0.1407	0.1396	0.1306	0.1376	0.1366	0.1356	0.1346	0.1336	0.1326
2.7	0.1317	0.1307	0.1298	0.1289	0.1279	0.1270	0.1261	0.1253	0.1244	0.1235
2.8	0.1227	0.1216	0.1210	0.1201	0.1193	0.1185	0.1177	0.1169	0.1161	0.1153
2.9	0.1145	0.1138	0.1130	0.1123	0.1115	0.1108	0.1100	0.1093	0.1086	0.1079
3.0	0.1072	0.1065	0.1058	0.1051	0.1045	0.1038	0.1031	0.1025	0.1018	0.1012
3.1	0.1005	0.0999	0.0993	0.0987	0.0981	0.0975	0.0969	0.0963	0.0957	0.0951
3.2	0.0945	0.0939	0.0934	0.0928	0.0922	0.0917	0.0911	0.0906	0.0901	0.0895
3.3	0.0890	0.0885	0.0880	0.0874	0.0869	0.0864	0.0859	0.0854	0.0849	0.0844
3.4	0.0839	0.0834	0.0830	0.0825	0.0820	0.0815	0.0811	0.0806	0.0802	0.0797
3.5	0.0793	0.0788	0.0784	0.0779	0.0775	0.0771	0.0767	0.0762	0.0758	0.0754
3.6	0.0750									

"a" Curve

$\bar{\lambda}$	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.2	1.0000	0.9981	0.9962	0.9942	0.9922	0.9900	0.9877	0.9854	0.9829	0.9805
0.3	0.9780	0.9756	0.9731	0.9706	0.9682	0.9657	0.9632	0.9607	0.9582	0.9556
0.4	0.9530	0.9504	0.9477	0.9449	0.9421	0.9392	0.9362	0.9330	0.9298	0.9265
0.5	0.9230	0.9193	0.9156	0.9117	0.9078	0.9039	0.9000	0.8961	0.8923	0.8885
0.6	0.8848	0.8810	0.8772	0.8733	0.8693	0.8652	0.8611	0.8570	0.8530	0.8489
0.7	0.8447	0.8404	0.8359	0.8312	0.8264	0.8214	0.8164	0.8115	0.8055	0.8015
0.8	0.7965	0.7914	0.7860	0.7806	0.7749	0.7692	0.7634	0.7575	0.7515	0.7455
0.9	0.7394	0.7333	0.7270	0.7207	0.7143	0.7078	0.7013	0.6947	0.6880	0.6813
1.0	0.6746	0.6678	0.6610	0.6541	0.6473	0.6404	0.6336	0.6267	0.6198	0.6130
1.1	0.6061	0.5993	0.5925	0.5858	0.5791	0.5725	0.5660	0.5595	0.5530	0.5466
1.2	0.5403	0.5339	0.5276	0.5213	0.5151	0.5090	0.5029	0.4970	0.4911	0.4854
1.3	0.4798	0.4742	0.4687	0.4633	0.4580	0.4527	0.4475	0.4423	0.4372	0.4321
1.4	0.4271	0.4221	0.4172	0.4124	0.4077	0.4030	0.3984	0.3939	0.3894	0.3850
1.5	0.3807	0.3764	0.3722	0.3681	0.3640	0.3600	0.3560	0.3521	0.3482	0.3444
1.6	0.3406	0.3369	0.3333	0.3297	0.3262	0.3227	0.3193	0.3159	0.3126	0.3094
1.7	0.3062	0.3031	0.3000	0.2970	0.2940	0.2910	0.2881	0.2852	0.2824	0.2796
1.8	0.2768	0.2741	0.2714	0.2687	0.2661	0.2635	0.2609	0.2583	0.2557	0.2532
1.9	0.2507	0.2482	0.2458	0.2434	0.2410	0.2387	0.2364	0.2342	0.2320	0.2298
2.0	0.2277	0.2256	0.2235	0.2215	0.2194	0.2174	0.2153	0.2133	0.2113	0.2094
2.1	0.2076	0.2056	0.2041	0.2024	0.2007	0.1990	0.1973	0.1956	0.1939	0.1923
2.2	0.1906	0.1890	0.1873	0.1857	0.1842	0.1826	0.1811	0.1795	0.1780	0.1766
2.3	0.1751	0.1737	0.1723	0.1709	0.1696	0.1682	0.1668	0.1655	0.1642	0.1628
2.4	0.1615	0.1602	0.1589	0.1576	0.1563	0.1551	0.1539	0.1527	0.1515	0.1503
2.5	0.1492	0.1482	0.1471	0.1461	0.1449	0.1437	0.1425	0.1414	0.1404	0.1394
2.6	0.1384	0.1373	0.1362	0.1351	0.1341	0.1332	0.1323	0.1313	0.1303	0.1294
2.7	0.1285	0.1275	0.1266	0.1256	0.1247	0.1238	0.1229	0.1220	0.1212	0.1203
2.8	0.1195	0.1187	0.1179	0.1171	0.1163	0.1155	0.1147	0.1140	0.1132	0.1124
2.9	0.1117	0.1110	0.1103	0.1096	0.1089	0.1082	0.1075	0.1068	0.1061	0.1055
3.0	0.1048	0.1041	0.1035	0.1028	0.1022	0.1015	0.1008	0.1002	0.0995	0.0988
3.1	0.0982	0.0976	0.0970	0.0964	0.0958	0.0952	0.0945	0.0940	0.0935	0.0929
3.2	0.0923	0.0917	0.0912	0.0906	0.0901	0.0895	0.0889	0.0884	0.0878	0.0873
3.3	0.0868	0.0863	0.0858	0.0854	0.0849	0.0844	0.0839	0.0834	0.0829	0.0824
3.4	0.0819	0.0814	0.0810	0.0806	0.0801	0.0797	0.0793	0.0788	0.0784	0.0779
3.5	0.0775	0.0771	0.0766	0.0762	0.0758	0.0754	0.0750	0.0746	0.0742	0.0738
3.6	0.0734									

"b" Curve

$\bar{\lambda}$	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.2	1.0000	0.9967	0.9933	0.9899	0.9865	0.9830	0.9795	0.9760	0.9724	0.9687
0.3	0.9650	0.9612	0.9573	0.9533	0.9493	0.9453	0.9412	0.9372	0.9331	0.9291
0.4	0.9250	0.9211	0.9171	0.9132	0.9093	0.9054	0.9014	0.8974	0.8933	0.8892
0.5	0.8850	0.8807	0.8762	0.8717	0.8671	0.8624	0.8577	0.8529	0.8480	0.8430
0.6	0.8380	0.8329	0.8278	0.8227	0.8174	0.8122	0.8068	0.8015	0.7960	0.7905
0.7	0.7850	0.7794	0.7738	0.7681	0.7624	0.7566	0.7508	0.7449	0.7390	0.7330
0.8	0.7270	0.7210	0.7148	0.7087	0.7024	0.6961	0.6897	0.6832	0.6766	0.6700
0.9	0.6633	0.6566	0.6500	0.6434	0.6369	0.6305	0.6241	0.6177	0.6114	0.6051
1.0	0.5987	0.5924	0.5861	0.5799	0.5737	0.5676	0.5615	0.5554	0.5495	0.5435
1.1	0.5376	0.5318	0.5260	0.5202	0.5145	0.5088	0.5031	0.4975	0.4919	0.4864
1.2	0.4809	0.4754	0.4700	0.4647	0.4593	0.4541	0.4489	0.4438	0.4387	0.4337
1.3	0.4288	0.4240	0.4192	0.4145	0.4098	0.4052	0.4007	0.3962	0.3918	0.3874
1.4	0.3831	0.3788	0.3746	0.3704	0.3663	0.3622	0.3582	0.3542	0.3503	0.3464
1.5	0.3426	0.3389	0.3352	0.3317	0.3281	0.3246	0.3212	0.3178	0.3144	0.3111
1.6	0.3078	0.3046	0.3014	0.2982	0.2950	0.2919	0.2888	0.2857	0.2826	0.2796
1.7	0.2766	0.2737	0.2709	0.2681	0.2654	0.2617	0.2601	0.2576	0.2551	0.2526
1.8	0.2502	0.2478	0.2455	0.2431	0.2408	0.2385	0.2362	0.2340	0.2317	0.2295
1.9	0.2273	0.2251	0.2230	0.2208	0.2188	0.2167	0.2147	0.2127	0.2108	0.2089
2.0	0.2070	0.2052	0.2034	0.2016	0.1999	0.1982	0.1965	0.1948	0.1931	0.1914
2.1	0.1897	0.1880	0.1864	0.1848	0.1833	0.1818	0.1804	0.1790	0.1776	0.1761
2.2	0.1746	0.1730	0.1715	0.1701	0.1688	0.1675	0.1662	0.1648	0.1625	0.1621
2.3	0.1607	0.1594	0.1580	0.1567	0.1555	0.1542	0.1530	0.1518	0.1506	0.1494
2.4	0.1483	0.1471	0.1460	0.1449	0.1438	0.1427	0.1417	0.1407	0.1397	0.1387
2.5	0.1377	0.1366	0.1356	0.1346	0.1336	0.1327	0.1319	0.1311	0.1303	0.1293
2.6	0.1283	0.1273	0.1263	0.1253	0.1244	0.1237	0.1230	0.1222	0.1214	0.1206
2.7	0.1198	0.1190	0.1182	0.1174	0.1166	0.1158	0.1150	0.1142	0.1134	0.1127
2.8	0.1119	0.1111	0.1104	0.1096	0.1088	0.1081	0.1074	0.1066	0.1059	0.1052
2.9	0.1045	0.1038	0.1031	0.1024	0.1017	0.1010	0.1003	0.0997	0.0990	0.0983
3.0	0.0977	0.0971	0.0964	0.0958	0.0951	0.0945	0.0939	0.0932	0.0926	0.0920
3.1	0.0914	0.0908	0.0902	0.0896	0.0891	0.0885	0.0879	0.0874	0.0868	0.0863
3.2	0.0857	0.0852	0.0846	0.0841	0.0835	0.0830	0.0825	0.0819	0.0814	0.0809
3.3	0.0804	0.0799	0.0794	0.0789	0.0784	0.0779	0.0774	0.0769	0.0764	0.0760
3.4	0.0755	0.0750	0.0746	0.0742	0.0737	0.0733	0.0729	0.0724	0.0720	0.0716
3.5	0.0712	0.0708	0.0704	0.0700	0.0697	0.0693	0.0689	0.0686	0.0682	0.0679
3.6	0.0675									

"c" Curve

$\bar{\lambda}$	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.2	1.0000	0.9949	0.9899	0.9849	0.9799	0.9750	0.9702	0.9654	0.9606	0.9558
0.3	0.9510	0.9461	0.9412	0.9362	0.9312	0.9261	0.9210	0.9158	0.9106	0.9053
0.4	0.9000	0.8947	0.8893	0.8838	0.8783	0.8727	0.8671	0.8613	0.8555	0.8496
0.5	0.8436	0.8376	0.8316	0.8256	0.8196	0.8136	0.8076	0.8015	0.7954	0.7892
0.6	0.7829	0.7766	0.7701	0.7636	0.7571	0.7506	0.7441	0.7377	0.7314	0.7250
0.7	0.7187	0.7124	0.7060	0.6997	0.6933	0.6869	0.6804	0.6738	0.6673	0.6608
0.8	0.6543	0.6478	0.6416	0.6353	0.6292	0.6232	0.6171	0.6111	0.6051	0.5991
0.9	0.5931	0.5871	0.5812	0.5754	0.5696	0.5640	0.5584	0.5529	0.5474	0.5421
1.0	0.5368	0.5315	0.5263	0.5211	0.5159	0.5108	0.5057	0.5006	0.4956	0.4906
1.1	0.4856	0.4807	0.4758	0.4710	0.4662	0.4614	0.4567	0.4521	0.4471	0.4428
1.2	0.4383	0.4338	0.4293	0.4249	0.4205	0.4162	0.4119	0.4076	0.4034	0.3993
1.3	0.3952	0.3911	0.3871	0.3832	0.3792	0.3754	0.3715	0.3678	0.3640	0.3604
1.4	0.3567	0.3532	0.3496	0.3462	0.3427	0.3393	0.3360	0.3328	0.3295	0.3263
1.5	0.3232	0.3211	0.3170	0.3139	0.3109	0.3078	0.3048	0.3018	0.2989	0.2959
1.6	0.2930	0.2900	0.2871	0.2842	0.2813	0.2785	0.2758	0.2731	0.2704	0.2678
1.7	0.2652	0.2626	0.2600	0.2575	0.2550	0.2525	0.2501	0.2478	0.2455	0.2432
1.8	0.2410	0.2388	0.2366	0.2345	0.2324	0.2303	0.2282	0.2262	0.2242	0.2222
1.9	0.2203	0.2184	0.2165	0.2146	0.2128	0.2110	0.2092	0.2075	0.2058	0.2041
2.0	0.2024	0.2007	0.1991	0.1974	0.1958	0.1942	0.1926	0.1910	0.1895	0.1879
2.1	0.1864	0.1850	0.1837	0.1823	0.1807	0.1790	0.1774	0.1759	0.1745	0.1731
2.2	0.1718	0.1703	0.1688	0.1674	0.1662	0.1650	0.1637	0.1624	0.1611	0.1598
2.3	0.1585	0.1572	0.1560	0.1548	0.1536	0.1524	0.1512	0.1501	0.1489	0.1478
2.4	0.1467	0.1456	0.1445	0.1435	0.1424	0.1414	0.1404	0.1394	0.1385	0.1375
2.5	0.1366	0.1357	0.1347	0.1337	0.1328	0.1318	0.1308	0.1300	0.1292	0.1283
2.6	0.1273	0.1261	0.1250	0.1244	0.1237	0.1230	0.1222	0.1214	0.1205	0.1196
2.7	0.1188	0.1181	0.1173	0.1165	0.1158	0.1150	0.1142	0.1135	0.1128	0.1120
2.8	0.1113	0.1106	0.1098	0.1091	0.1084	0.1077	0.1070	0.1063	0.1056	0.1050
2.9	0.1043	0.1036	0.1030	0.1023	0.1017	0.1010	0.1003	0.0997	0.0990	0.0984
3.0	0.0977	0.0971	0.0964	0.0958	0.0951	0.0945	0.0939	0.0932	0.0926	0.0920
3.1	0.0914	0.0908	0.0902	0.0896	0.0891	0.0885	0.0879	0.0874	0.0868	0.0863
3.2	0.0857	0.0852	0.0846	0.0841	0.0835	0.0830	0.0825	0.0819	0.0814	0.0809
3.3	0.0804	0.0799	0.0794	0.0789	0.0784	0.0779	0.0774	0.0769	0.0764	0.0760
3.4	0.0755	0.0750	0.0746	0.0742	0.0737	0.0733	0.0729	0.0724	0.0720	0.0716
3.5	0.0712	0.0708	0.0704	0.0700	0.0697	0.0693	0.0689	0.0686	0.0682	0.0679
3.6	0.0675									

"d" Curve

$\bar{\lambda}$	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.1	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
0.2	1.0000	0.9916	0.9829	0.9742	0.9656	0.9570	0.9487	0.9405	0.9325	0.9247
0.3	0.9170	0.9093	0.9017	0.8941	0.8866	0.8790	0.8713	0.8637	0.8560	0.8483
0.4	0.8407	0.8332	0.8259	0.8187	0.8115	0.8044	0.7974	0.7903	0.7833	0.7762
0.5	0.7691	0.7620	0.7549	0.7478	0.7407	0.7336	0.7266	0.7196	0.7126	0.7057
0.6	0.6989	0.6921	0.6853	0.6786	0.6719	0.6653	0.6587	0.6522	0.6457	0.6393
0.7	0.6329	0.6265	0.6202	0.6140	0.6078	0.6017	0.5957	0.5897	0.5837	0.5776
0.8	0.5720	0.5662	0.5605	0.5549	0.5493	0.5438	0.5383	0.5329	0.5276	0.5223
0.9	0.5171	0.5119	0.5067	0.5018	0.4988	0.4919	0.4870	0.4821	0.4774	0.4727
1.0	0.4681	0.4635	0.4589	0.4544	0.4500	0.4456	0.4413	0.4370	0.4328	0.4286
1.1	0.4244	0.4204	0.4163	0.4123	0.4084	0.4045	0.4006	0.3968	0.3930	0.3892
1.2	0.3855	0.3819	0.3782	0.3746	0.3711	0.3676	0.3641	0.3606	0.3972	0.3538
1.3	0.3505	0.3472	0.3439	0.3407	0.3375	0.3343	0.3312	0.3281	0.3250	0.3219
1.4	0.3189	0.3159	0.3130	0.3101	0.3072	0.3043	0.3016	0.2987	0.2959	0.2932
1.5	0.2905	0.2878	0.2862	0.2826	0.2800	0.2774	0.2749	0.2724	0.2700	0.2675
1.6	0.2651	0.2627	0.2603	0.2580	0.2557	0.2534	0.2511	0.2489	0.2467	0.2445
1.7	0.2423	0.2402	0.2381	0.2360	0.2339	0.2319	0.2299	0.2279	0.2259	0.2239
1.8	0.2220	0.2201	0.2182	0.2163	0.2145	0.2126	0.2100	0.2090	0.2073	0.2055
1.9	0.2038	0.2021	0.2004	0.1988	0.1971	0.1955	0.1939	0.1923	0.1907	0.1891
2.0	0.1876	0.1861	0.1846	0.1831	0.1816	0.1802	0.1787	0.1773	0.1759	0.1745
2.1	0.1731	0.1717	0.1704	0.1691	0.1677	0.1664	0.1651	0.1639	0.1626	0.1614
2.2	0.1601	0.1589	0.1577	0.1565	0.1553	0.1542	0.1530	0.1519	0.1507	0.1496
2.3	0.1485	0.1474	0.1463	0.1452	0.1442	0.1431	0.1421	0.1410	0.1400	0.1390
2.4	0.1380	0.1370	0.1361	0.1351	0.1341	0.1332	0.1322	0.1313	0.1304	0.1295
2.5	0.1286	0.1277	0.1268	0.1259	0.1251	0.1242	0.1234	0.1225	0.1217	0.1209
2.6	0.1201	0.1193	0.1185	0.1177	0.1169	0.1161	0.1153	0.1146	0.1138	0.1131
2.7	0.1123	0.1116	0.1109	0.1101	0.1094	0.1087	0.1080	0.1073	0.1066	0.1059
2.8	0.1052	0.1045	0.1038	0.1031	0.1024	0.1018	0.1011	0.1004	0.0998	0.0991
2.9	0.0985	0.0978	0.0972	0.0965	0.0959	0.0952	0.0946	0.0940	0.0934	0.0927
3.0	0.0921									

Annex 5: Buckling length of compression members

A5.1 General

The buckling length L_{cr} of a compression member is the length of otherwise r similar member with “hinged ends” (ends that cannot be laterally displaced, but which are free to gyrate on the plane of buckling), which has the same buckling resistance.

Without more information, the theoretical elastic buckling length may be conservatively adopted as buckling length.

An equivalent buckling length may be used to relate the buckling resistance of a member subject to a non-uniform axial force to that of a similar member subject to a uniform axial force.

An equivalent buckling length may also be used to relate the buckling resistance of a non-constant cross section to that of another uniform member subject to similar conditions of force and of linking.

A5.2 Structural supports or frames in buildings

The buckling length L_{cr} of a support in a non-translational frame (fixed joint type) can be seen in figure A5.2.a.

The buckling length L_{cr} of a support in a translational frame (moving joint type) can be seen in figure A5.2.b.

For the theoretical models shown in figure A5.2.c, the distribution coefficients η_1 and η_2 are obtained from:

$$\eta_1 = K_c/(K_c + K_{11} + K_{12})$$

$$\eta_2 = K_c/(K_c + K_{21} + K_{22})$$

where:

K_c Coefficient of stiffness of column I/L.

K_{ij} Effective coefficient of sitffness of the beam.

These models can be adapted for the sizing of continuous supports, assuming that each longitudinal section of the support is stressed to the same value of the ratio (N/N_{cr}). In the general case in which (N/N_{cr}) varies, this leads to a conservative value of L_{cr}/L for the most critical length of the column.

For each longitudinal section of a continuous support, the hypothesis mentioned in the previous paragraph may be considered, using the model indicated in figure A5.2.d and obtaining the distribution coefficients η_1 and η_2 from:

$$\eta_1 = (K_c + K_1) / (K_c + K_1 + K_{11} + K_{12})$$

$$\eta_2 = (K_c + K_2) / (K_c + K_2 + K_{21} + K_{22})$$

where K_1 and K_2 are the coefficients of sitffness for the longitudinal sections adjacent to the support.

Where the beams are not subject to axial forces, their coefficients of sitffness may be determined according to table A5.2.a, provided that they fall within the elastic range.

Table A5.2.a. Effective coefficient of sitffness for a beam

Conditions of restraint to rotation on the far end of the beam	Effective coefficient of sitffness K of the beam (provided this remains within the elastic range)
Fixed on the far end	1.0 I/L
Hinged on the far end	0.75 I/L
Equal rotation to that of the near end (double curvature)	1.5 I/L
Equal and opposite rotation to that of the near end (single curvature)	0.5 I/L
General case. Rotation θ_a on the near end and θ_b on the far end	$(1 + 0.5\ \theta_b/\theta_a)$ I/L

For building frames with concrete floor slabs, provided that the frame or structure is of a regular geometry and bears a uniform load, it is generally sufficiently accurate to assume that the coefficients of sitffness of the beams are those indicated in table A5.2.b.

Table A5.2.b. Effective coefficient of sitffness for beams of a building frame with concrete floor slab

Load conditions for the beam	Non-translational frame	Translational frame
Beams directly supporting concrete floor slabs	1.0 I/L	1.0 I/L
Other beams with direct loads	0.75 I/L	1.0 I/L
Beams with moments on the ends only	0.5 I/L	1.5 I/L

When, for the same load, the design value of the bending moment on any of the beams exceeds the value $W_{el}f_y / \gamma_{M0}$, it must be assumed that the beam is hinged at the corresponding point(s).

Where a beam has nominally articulated joints, it shall be assumed that it is hinged at the corresponding point(s).

Where a beam contains semi-rigid joints, its effective coefficient of sitffness must be reduced accordingly.

Figure A5.2.a. Buckling length ratio L_{cr}/L (coefficient β) for a non-translational frame support (fixed joints)

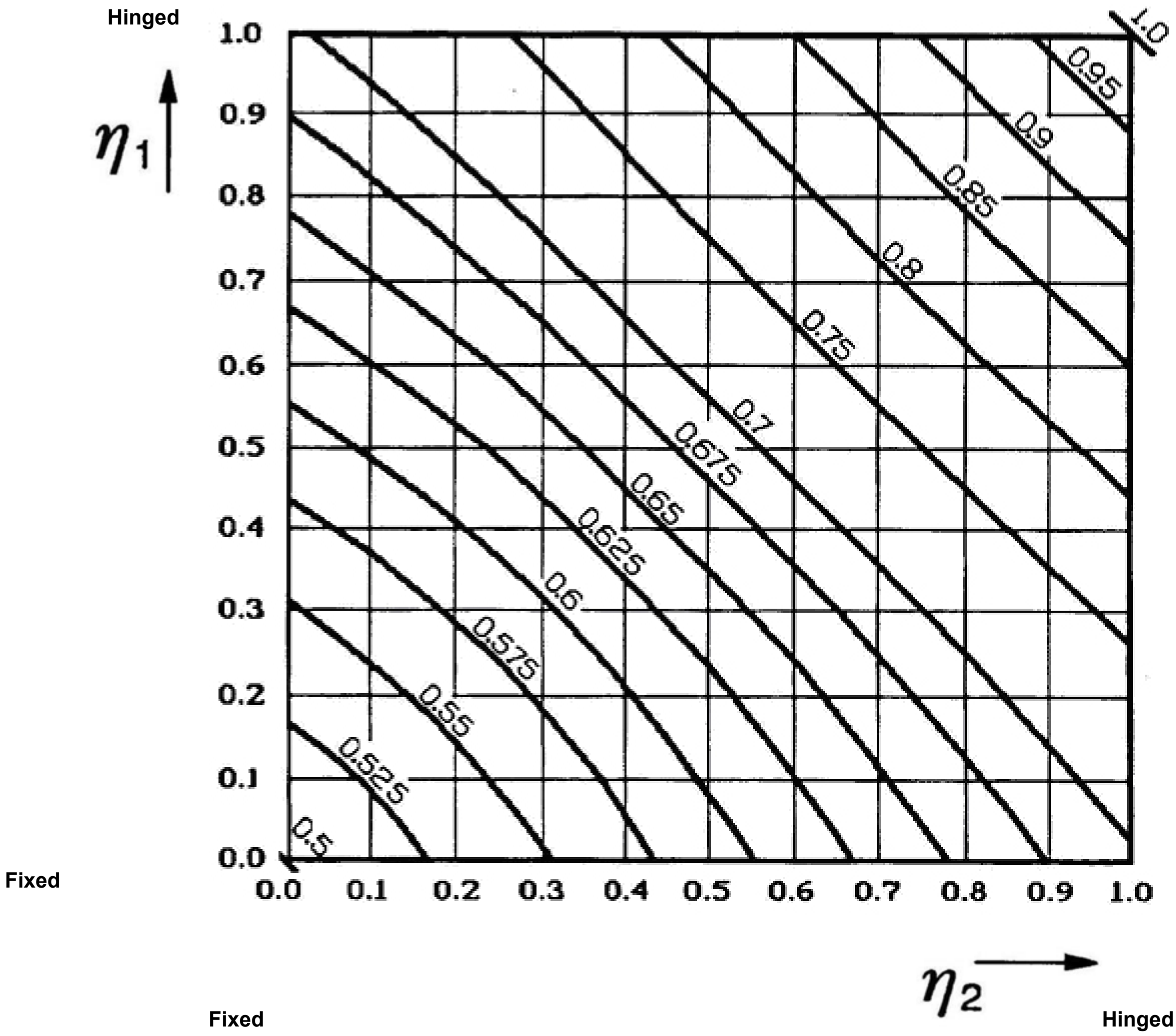


Figure A5.2.a. Buckling length ratio L_{cr}/L (coefficient β) for a non-translational frame support (fixed joints)

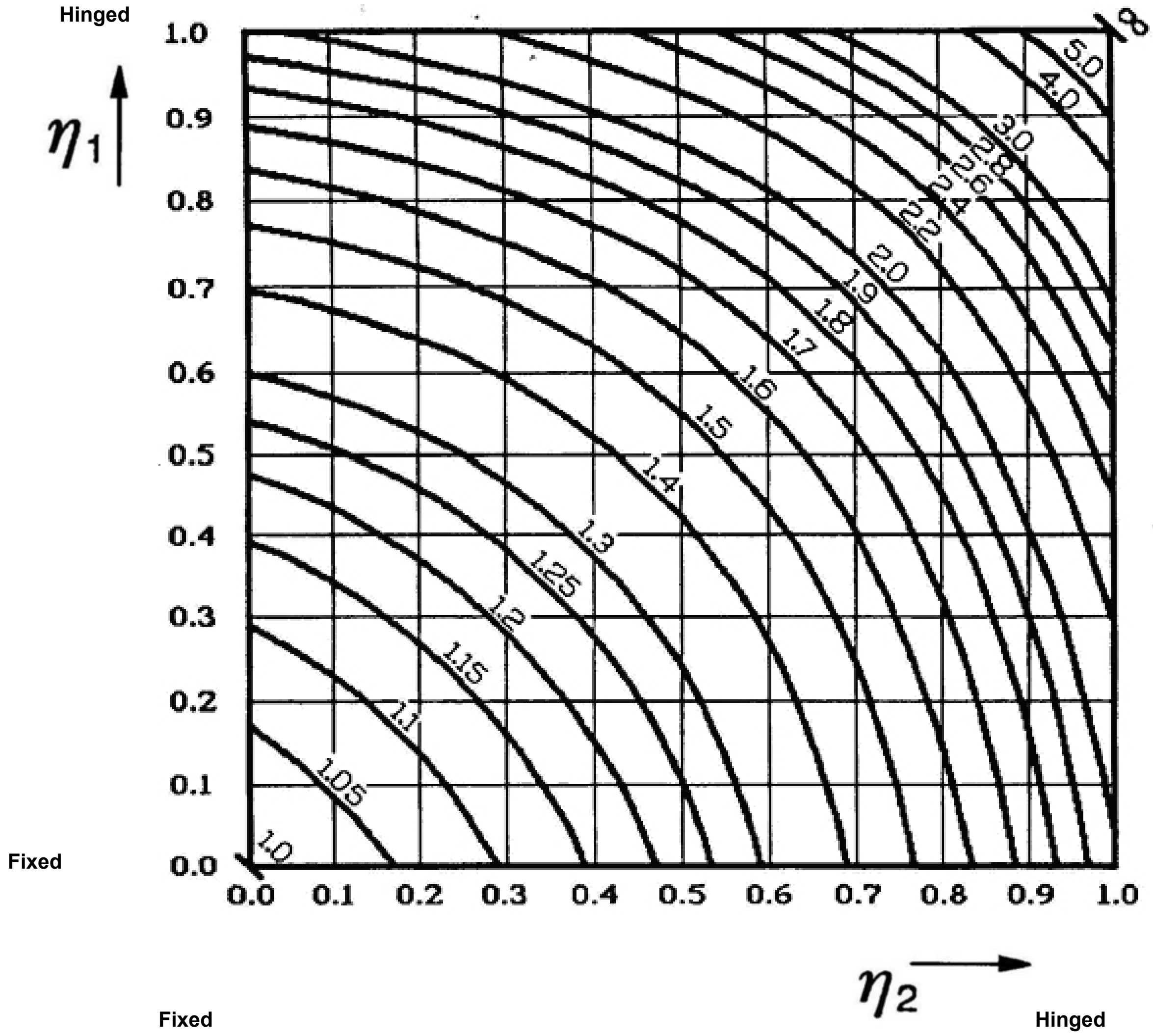
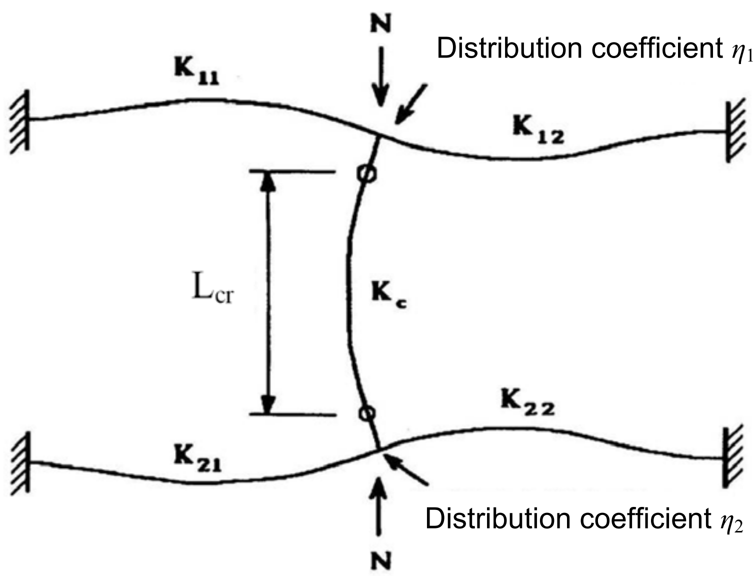
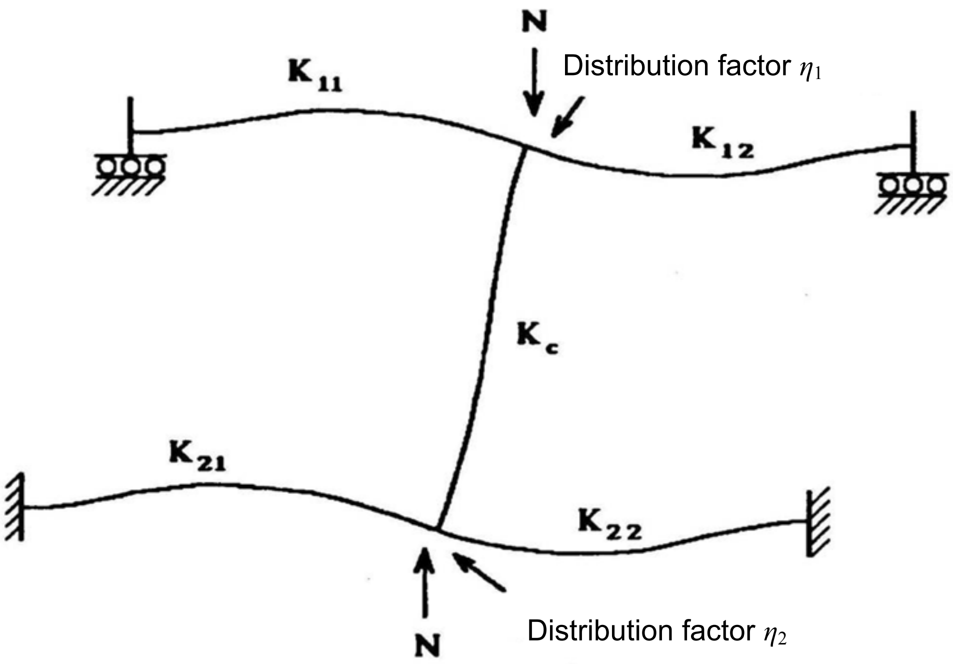


Figure A5.2.b. Buckling length ratio L_{cr}/L (coefficient β) for a translational frame support (moving joints)

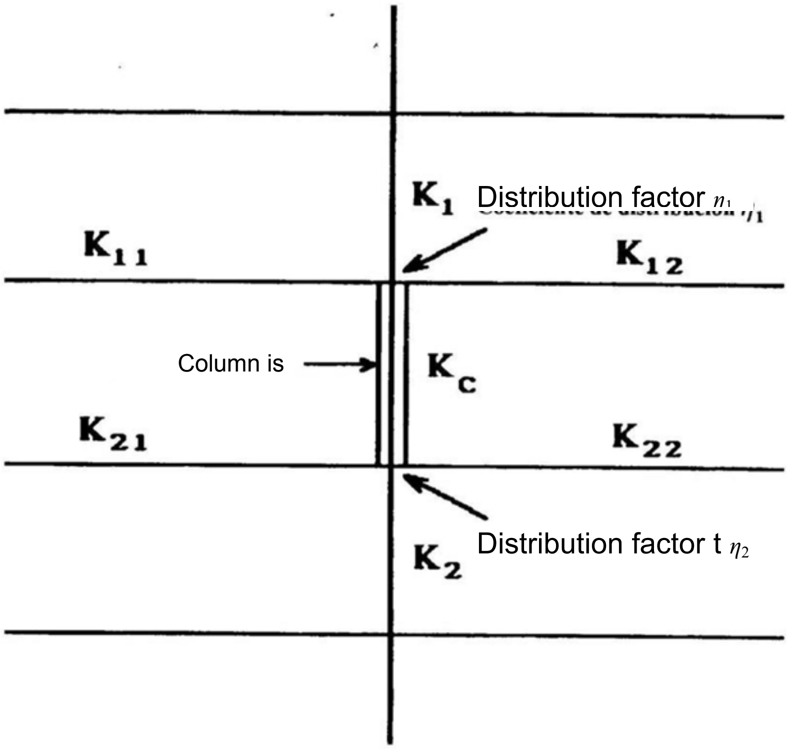


a) Non-translational mode (buckling with fixed joints)



b) Translational mode (buckling with moving joints)

Figure A5.2.c. Distribution factors for supports



$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}}$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}}$$

Figure A5.2.d. Distribution factors for continuous supports

Where beams are loaded due axial force, their effective stiffness factors must be adjusted accordingly. This can be done using stability functions. As a simple alternative, the increase in the stiffness factors due to the existence of a tensile axial force may be disregarded with consideration given instead to the existence of a compression axial force using the conservative approximations shown in table A5.2.c.

Table A5.2.c. Approximate formulae for bend stiffness factors, reduced due to the existence of compression axial force

Conditions of restraint on the far end of the beam	Effective stiffness factor K of the beam (providing this remains within the elastic range)
Fixed on the far end	1.0 I/L (1 – 0.4 N/N_E)
Articulated on the far end	0.75 I/L (1 – 1.0 N/N_E)
Equal rotation to that of the near end (double curvature)	1.5 I/L (1 – 0.2 N/N_E)
Equal and opposite rotation to that of the near end (single curvature)	0.5 I/L (1 – 1.0 N/N_E)

In this table, N_E = ð$_2$EI/L_2

The empirical formulae given below may be used as conservative estimates instead of the values taken from figures A5.2.a and A5.2.b.

a) Non-translational mode (figure A5.2.a):

$$\frac{L_{cr}}{L} = 0,5 + 0,14(\eta_1 + \eta_2) + 0,055(\eta_1 + \eta_2)^2$$

a) Translational mode (figure A5.2.b):

$$\frac{L_{cr}}{L} = \sqrt{\frac{1 - 0,2(\eta_1 + \eta_2) - 0,12\eta_1\eta_2}{1 - 0,8(\eta_1 + \eta_2) + 0,6\eta_1\eta_2}}$$

Annex 6: Longitudinally stiffened plate elements

A6.1 General

This Annex gives rules for taking into account the ultimate limit state for local buckling in stiffened plate elements, caused by longitudinal stresses, when the following conditions are met:

- The panels are rectangular and the flanges are parallel, or near-parallel with a maximum inclination of 10º.
- Where stiffeners are present, they are configured in the longitudinal and/or transverse direction.
- Any holes or cuts must be small in size.
- The structural elements in question are of a uniform transverse section.
- There is no local buckling in the web generated by the flange.

A6.2 Resistance to longitudinal stresses

The resistance of stiffened plate structural elements subject to longitudinal stresses can be determined using the effective sections of compressed plate members used to calculate the sectional properties of a grade 4 section (A_{ef}, I_{ef}, W_{ef}), with the aim of testing the resistance of the sections and members to buckling and lateral buckling, in accordance with sections 35.1 and 35.2 of Chapter IX.

Effective sections can be determined based on the indications given in Section 20, in particular section 20.7, of Chapter V. Consideration must also be given to the influence of shear lag. Considering both effects, effective sections are determined in accordance with the provisions of sections 21.5 and 34.1.2.5.

A6.3 Plate elements without longitudinal stiffening

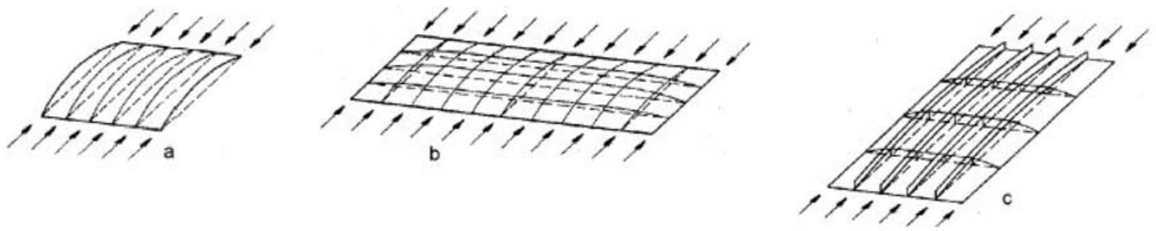
The effective areas of compression flat elements are defined in tables 20.7.a and 20.7.b of section 20.7 for members without free ends (interior panels) and for members with a free end, respectively. Except for ultimate limit state testing on members prone to instability issues, as addressed in Chapter IX sections 35.1, 35.2 and 35.3, the effective widths of grade 4 compressed cross section panels may be estimated, in a less conservative manner, based on a lower plate slenderness value $_{p}\lambda$ calculated on the basis of maximum compressed panel stress or warping values, which are obtained considering the effective widths of all partial or fully compressed cross section panels.

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\frac{\sigma_{c,Ed}}{f_y / \gamma_{M0}}} < \overline{\lambda}_p$$

In this expression $\sigma_{c,Ed}$ is the maximum design compression stress on the plate element , determined using the effective area of the section, taking into account all actions applied simultaneously.

The procedure described in the previous paragraph is conservative and requires an iterative design process, as indicated in Chapter V, section 20.7.

Sheet panels with a/b<1 aspect ratios (a being the distance between loaded ends) may be susceptible to column buckling. Testing is therefore carried out according to the indications given in section A6.4.4 of this Annex, using reduction coefficient ρ_c of panel instability (plate performance and column buckling performance). This is the case for flat sheet elements between transverse stiffeners in which plate instability can be associated with column buckling, requiring reduction coefficient ρ_c, in order to obtain the value χ_c for support buckling (see figures A6.3.a and A6.3.b). For sheets with longitudinal stiffeners, column buckling may also be seen for a/b aspect ratios equal to or greater than the unit.



a) plates without longitudinal supports b) nun-stiffened plate with small aspect ratio c) of longitudinally stiffened plate with a large aspect ratio

Figure A6.3. Column-like performance

A6.4 Plate elements with longitudinal stiffening

A6.4.1 General

For longitudinally stiffened plate elements, consideration must be given to the effective section areas of the different subpanels between stiffeners in terms of local buckling and to the effective section area of the stiffened panel in relation to general buckling.

The effective area of each subpanel should be determined by means of a reduction factor according to the indications given in section A6.3 (see section 20.7), in order to account for local plate buckling. The same goes for subpanels in which the longitudinal stiffeners can be broken down (however, these tend to be designed as grade 1 or 2, with which the local buckling reduction factor is equal to the unit).

The stiffened panel, taking into account any effective areas of stiffeners, must be checked for general buckling (e.g. by considering the panel as an equivalent orthotropic plate), identifying a reduction coefficient ρ_c corresponding to the local buckling verification of the panel, as a whole.

The effective section of the compression zone of the stiffened panel should be taken as $A_{c,ef} = \rho_c \cdot A_{c,ef,loc} + \sum b_{bor,ef} t$

where $A_{c,ef,loc}$ is the sum of effective areas of all stiffeners and subpanels fully or partially situated in the compressed area, with the exception of effective parts supported by an adjacent sheet panel with a thickness $b_{bor,ef}$, as shown in the example given in figure A6.4.1.

The area $A_{c,ef,loc}$ should be obtained from :

$$A_{c,ef,loc} = A_{s\ell,ef} + \sum_c \rho_{loc} b_{c,loc} t$$

where:

$\sum_c$ applies to the width of the compressed stiffened panel, except the parts $b_{bor,ef}$.

$A_{sl,ef}$ Sum of the effective areas of all longitudinal stiffeners with a gross area A_{sl} situated in the compression zone , calculated according to the indications given in section A6.3 of this Annex (see section 20.7).

$b_{c,loc}$ Width of the compressed part of each subpanel.

ρ_{loc} Reduction factor for each subpanel calculated according to A6.3 (see section 20.7).

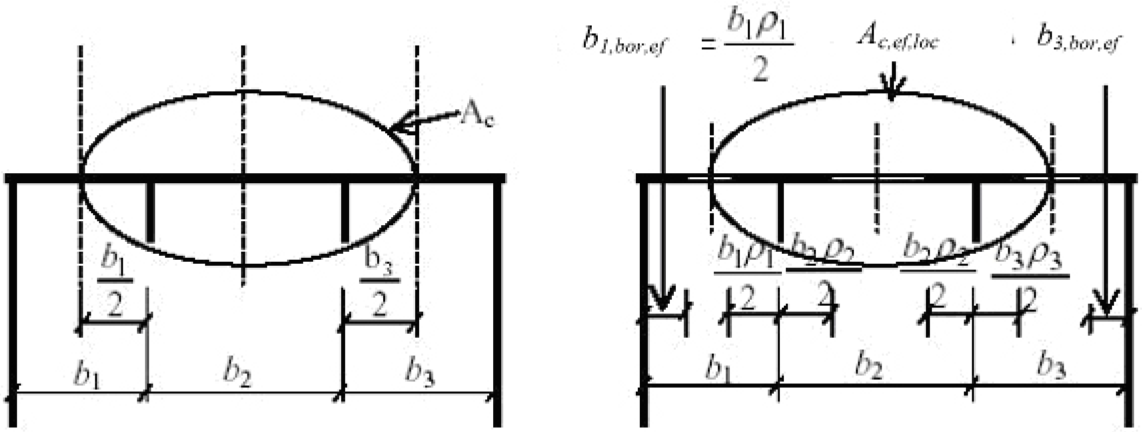


Figure A6.4.1. Definition of the gross area Ac and of the effective area $A_{c,ef,loc}$ for stiffened plates under to uniform compression

To determine the general reduction factor ρ_c which considers the instability of the panel as a whole, consideration must be given to the possibility of column buckling, which produces a stricter reduction factor than that derivable from plate buckling.

The procedure for carrying out this verification and for determining ρ_c is based on an interpolation between the reduction factor ρ related to plate buckling and the reduction factor c χ_c for considering column buckling. This interpolation is defined further on in section A6.4.4.

In cases where there is a significant influence of shear lag (see sections 21.5 and 34.1.2.5), the effective definitive reduced section $A_{c,ef}$ of the compressed area of the stiffened plate must be taken as $A^*_{c,ef}$ in order to account for the effect of shear lag in addition to the effects of plate buckling.

The effective section of the tensile area of the stiffened plate must be equal to the area of the gross section of the effective tensile area as a result of shear lag, where applicable.

The shear modulus W_{ef} of the effective reduced section is obtained as the quotient between the inertia of the effective reduced section and the distance between the centre of gravity of this section and the mid plane of the flange.

A6.4.2 Plate performance

The non-dimensional plate slenderness $\bar{\lambda}_p$ of the equivalent plate is defined as:

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,p}}}$$

where:

$$\beta_{A,c} = \frac{A_{c,ef,loc}}{A_c}$$

where:

A_c Gross section of the compressed area of the stiffened plate, with the exception of parts of subpanels supported by an adjacent plate panel (see figure A6.4.1) (which must be multiplied by the shear lag coefficient, where necessary).

$A_{c,ef,loc}$ Effective area of the same part of the plate, calculated taking into account the possible buckling of the different subpanels and/or flat stiffening elements.

The reduction coefficient ρ for an equivalent plate is obtained from the formula given for flat members without stiffening (see section 20.7 of Chapter V).

To determine the value $\bar{\lambda}_p$ using the aforementioned formula, it is necessary to know the stress value $\sigma_{cr,p}$. The two following subsections lay down two different methods for determining this stress value, respectively, each of which is particularly applicable to specific types of stiffened panels.

A6.4.2.1 Multiple longitudinal stiffeners. Concept of the equivalent orthotropic.

Stiffened plate panels with more than two longitudinal stiffeners may be considered orthotropic plate. The basic idea consists of the distribution of the stiffness of the longitudinal stiffeners across the width of the sheet. Conceptually, an equivalent orthotropic plate thus replaces the stiffened sheet. The critical buckling stress of the equivalent plate can be obtained as:

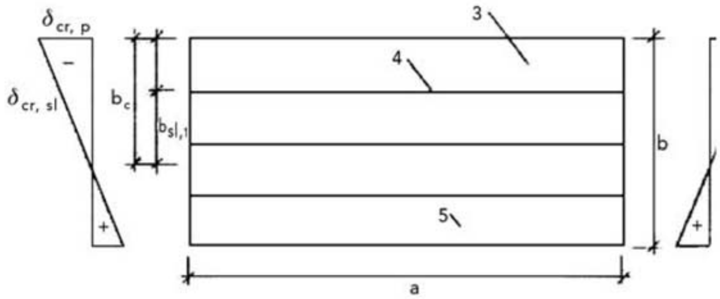
$$\sigma_{cr,p} = k_{\sigma,p}\sigma_E$$

where:

$$\sigma_E = \frac{\pi^2 E t^2}{12(1-\upsilon^2)b^2}$$

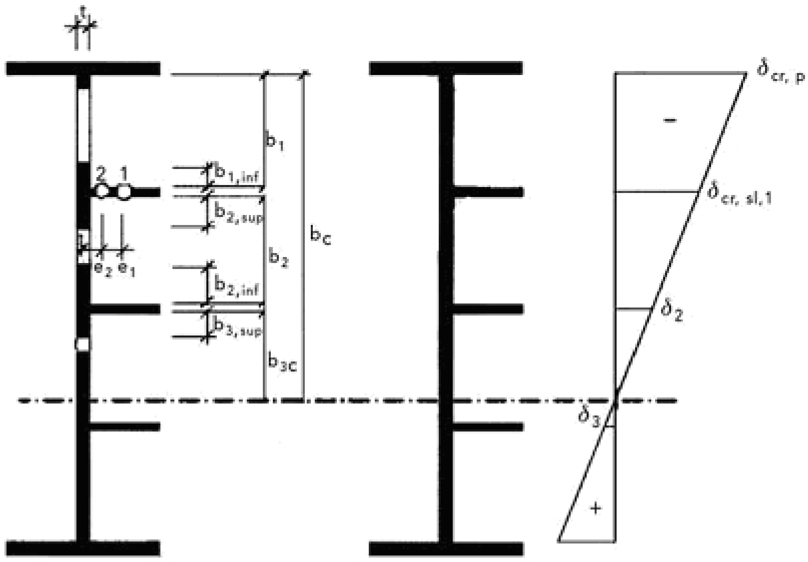
b Dimension of the panel subject to longitudinal stresses (see figure A6.4.2.1).

is the critical buckling stress on the edge of the panel where the maximum compression stress is applied (see figure A6.4.2.1).



1. centroid of the stiffener.
2. centroid of the column =stiffener+sheet.
3. Subpanel.
4. Stiffener.
5. Thickness of plate t t.

$e = \max(e_2 e_1)$



	Width for gross area	Width for effective area	Condition for Ψ_i
$b_{1,inf}$	$\frac{3-\psi_1}{5-\psi_1} b_1$	$\frac{3-\psi_1}{5-\psi_1} b_{1,ef}$	$\psi_1 = \frac{\sigma_{cr,sl,1}}{\sigma_{cr,p}} > 0$
$b_{2,sup}$	$\frac{2}{5-\psi_2} b_2$	$\frac{2}{5-\psi_2} b_{2,ef}$	$\psi_2 = \frac{\sigma_2}{\sigma_{cr,sl,1}} > 0$
$b_{2,inf}$	$\frac{3-\psi_2}{5-\psi_2} b_2$	$\frac{3-\psi_2}{5-\psi_2} b_{2,ef}$	$\psi_2 > 0$
$b_{3,sup}$	$0{,}4 b_{3c}$	$0{,}4 b_{3c,ef}$	$\psi_3 = \frac{\sigma_3}{\sigma_2} < 0$

Figure A6.4.2.1. Notation for longitudinally stiffened plates

The coefficient $k_{\sigma,p}$ for local buckling, obtained according to the orthotropic plate theory, considering longitudinal stiffeners distributed uniformly over the plate. This coefficient of local buckling of stiffened panels $k_{\sigma,p}$ is obtained either from appropriate charts for sneared stiffners or for discretely placed stiffeners, or relevant computer simulation.

Where a web is of concern, the width b in equation shown above should be replaced by the depth of the web h_w.

For stiffened panels with at least three equally spaced longitudinal stiffeners, the plate buckling coefficient $k_{\sigma,p}$ (global stiffened panel buckling) may be approximated by:

$$k_{\sigma,p} = \frac{2\left(\left(1+\alpha^2\right)^2 + \gamma - 1\right)}{\alpha^2(\psi+1)(1+\delta)} \qquad \text{if } \alpha \le \sqrt[4]{\gamma}$$

$$k_{\sigma,p} = \frac{4\left(1+\sqrt{\gamma}\right)}{(\psi+1)(1+\delta)} \qquad \text{if } \alpha \le \sqrt[4]{\gamma}$$

with:

$$\psi = \frac{\sigma_2}{\sigma_1} \ge 0{,}5$$

$$\gamma = \frac{I_{sl}}{I_p}$$

$$\delta = \frac{A_{sl}}{A_p}$$

$$\alpha = \frac{a}{b} \ge 0{,}5$$

where:

I_{sl} Second moment of area of the whole stiffened plate I_p: Second moment of area for bending of the plate:

$$I_p = \frac{bt^3}{12\left(1-\upsilon^2\right)} = \frac{bt^3}{10{,}92}$$

A_{sl} is the sum of the gross area of all individual longitudinal stiffeners.

A_p is the gross area of the plate:

Ap = b·t

σ_1 maximum stress at one panel edge.

σ_2 minimum stress at the opposite edge.

a, b and t Dimensions defined in figure A6.4.2.1.

A6.4.2.2 One or two stiffeners in compressed area. Concept of equivalent column on elastic foundation.

This procedure is of particular interest when the number and layout of longitudinal stiffeners result from a non-uniform distribution of direct longitudinal stresses, as in the case of a web panel. For such situations, following a procedure that addresses the discrete nature of stiffening in a simple manner is recommended. The critical buckling stress calculation can no longer be based on the orthotropic plate concept, but instead forms part of the analysis of a column rested on an elastic foundation, which reflects the plate effect in the direction perpendicular to the column bar (see figure A6.4.2.2.a). The critical elastic buckling stress of the equivalent column can be taken as an approximation of the stress value or $\sigma_{cr,p}$.

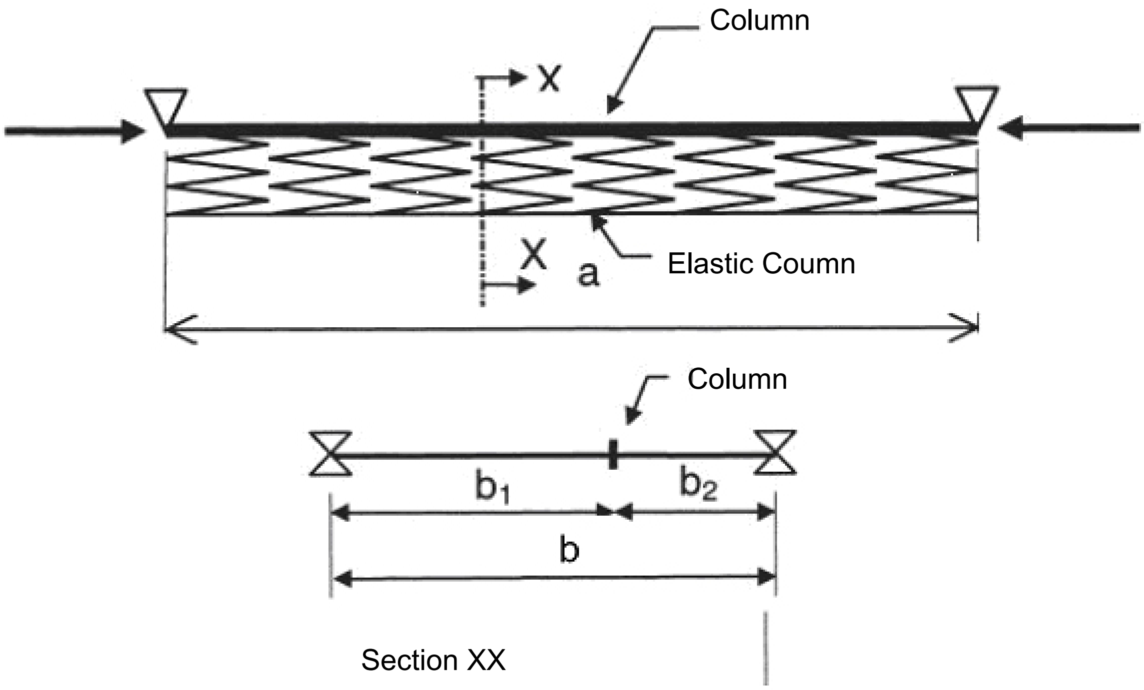


Figure A6.4.2.2.a. Column on elastic foundation

With a single stiffener:

If the stiffened plate contains just one stiffener in the compression zone, the localisation of the equivalent column coincides with the longitudinal stiffener. To ensure a simple formulation, stiffeners located in the tensile area of the member are not considered.

To calculate $A_{sl,1}$ and $I_{sl,1}$ the gross section of the column should t be taken as the gross area of the stiffener, considering the adjacent portions of the plate t as follows If the subpanel is fully in compression, a percentage of (3-Ψ)/(5-Ψ) of its width b_1 on the edge of the panel less stressed is taken, with a percentage 2/(5-Ψ) for the most stressed area. If the stress change from compression to tension) within the same subpanel, a portion of 0.4 of the width b_c of the compressed part of this subpanel should be taken as part of the column, as shown in figure A.6.4.2.2.b.

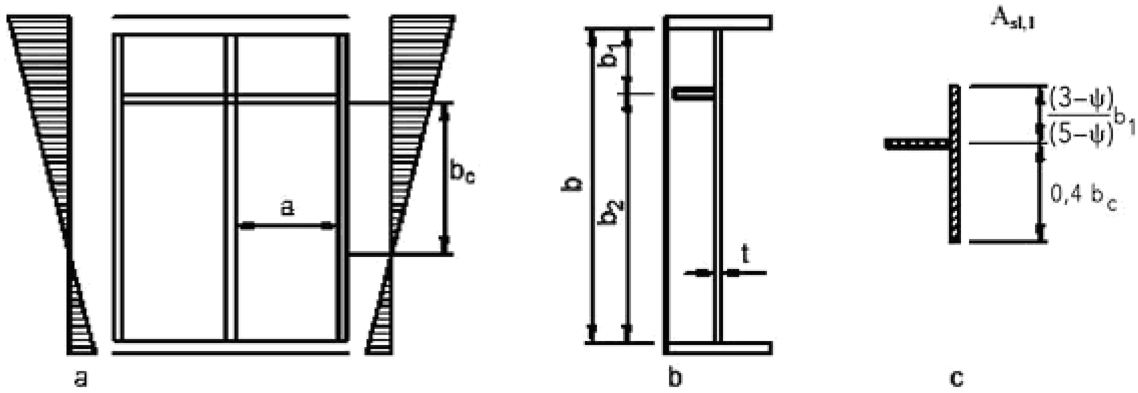


Figure A6.4.2.2.b. Notation for a web plate with a single stiffener in the compression zone The effective cross-sectional area of column $A_{sl,1,ef}$ should be taken as effective cross-sectional area of the stiffener and of the adjacent parts of the plate as seen in figure A6.4.2.1. The area of the effective section must be determined in order to calculate β_A. The slenderness of the plate elements in the column can be determined following the indications given in section 20.7, taking $\sigma_{c,Ed}$ as the maximum design compression stress for the gross cross section.

In absence of an elastic foundation, the buckling length of the equivalent column would be equal to the distance between transverse stiffeners *a*. Due to the plate effect, the buckling length a_c of the equivalent column would be smaller than distance *a*.

According to the physical model, it is found that:

$$a_c = 4{,}33\sqrt[4]{\frac{I_{sl,1}b_1^2b_2^2}{t^3b}}$$

The critical elastic buckling stress of the equivalent column can be determined using the following formulae:

$$\sigma_{cr,sl} = \frac{1{,}05\cdot E}{A_{sl,1}}\frac{\sqrt{I_{sl,1}t^3b}}{b_1b_2} \qquad \text{if } a \geq a_c$$

$$\sigma_{cr,sl} = \frac{\pi^2\cdot E\cdot I_{sl,1}}{A_{sl,1}\cdot a^2} + \frac{Et^3ba^2}{4\pi^2(1-\upsilon^2)A_{sl,1}b_1^2b_2^2} \qquad \text{if } a < a_c$$

where:

$A_{sl,1}$ Gross cross section of the column obtained.

$I_{sl,1}$ Second moment of area of the gross section of the column about an axis which passes through its centre of gravity and runs parallel to the plane of the sheet.

b_1, b_2 Distance from the longitudinal edges to the stiffener $b_1+b_2=b$ (see figures A6.4.2.2.a and A6.4.2.2.b).

This stress can be taken as an estimation of stress $\sigma_{cr,p}$.

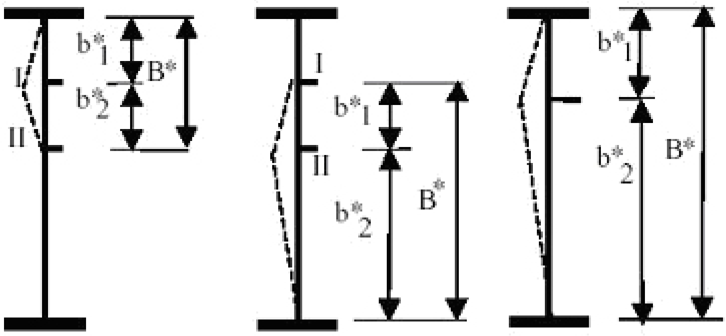
With two stiffeners:

Where the stiffened plate has two longitudinal stiffeners in the compression zone the method illustrated above is applied three times, as shown below (see figure A6.4.2.2.c). First, it is assumed that each of the stiffeners buckles while the other acts as a fixed support.

A simultaneous buckling verification must then be performed on both stiffeners; this involves using a single stiffener that combines the effect of both as follows:

- The cross-sectional area and second moment of area i of the combined stiffener are respectively the sum of that for the individual stiffeners, which are calculated as shown above.

- The combined stiffener is positioned at the location of the resultant of the respective forces in the individual stiffeners.



	Stiffener 1	Stiffener 2	Combined stiffener
Area	$A_{sl,1}$	$A_{sl,2}$	$A_{sl,1}+A_{sl,2}$
Second moment of area	$I_{sl,1}$	$I_{sl,2}$	$I_{sl,1} + I_{sl,2}$

Figure A6.4.2.2.c Notations for plate with two stiffeners in the compression zone
Consideration is given to the three situations shown in figure A6.4.2.2.c, assuming $b_1=b_1^*$, $b_2=b_2^*$ and $b = B^*=b_1^*+ b_2^*$.

The three cases are then studied and the smaller of the three values taken from $\sigma_{cr,sl}$, obtained using the formula shown in the previous section for the case of a single stiffener, is taken as an estimation of the stress value $\sigma_{cr,p}$.

A6.4.3 Column type buckling performance

The elastic critical buckling stress $\sigma_{cr,c}$ of a unstiffened plate (see section A6.3) or stiffened plates (see section A6.4) should be as the critical buckling stress without considering the supports along the longitudinal edges.

For a non-stiffened plate the critical elastic column buckling stress $\sigma_{cr,c}$ is obtained from :

$$\sigma_{cr,c} = \frac{\pi^2 E \cdot t^2}{12(1-\upsilon^2)a^2}$$

For a stiffened plate the stress $\sigma_{cr,c}$ may be determined based on the critical column buckling stress $\sigma_{cr,sl}$ of the stiffener closest to the panel edge with the highest compression stress, as follows

$$\sigma_{cr,sl} = \frac{\pi^2 E \cdot I_{sl,1}}{A_{sl,1} a^2}$$

where:

$I_{sl,1}$ the second moment of area of the gross cross section of the longitudinal stiffener and the adjacent parts of the plate, as indicated in figure A.6.4.2.1,relative to the out-of-plane bending axis of the plate $A_{sl,1}$ Gross cross sectional area of the stiffener and the adjacent parts of the plate, as indicated in figure A.6.4.2.1

The stress $\sigma_{cr,sl}$ is obtained considering a uniform compression stress state, without any influence from the outer end supports and with a buckling length equal to the length of the stiffened panel .

In the case of a non-uniform stress distribution, the stress $\sigma_{cr,c}$ can be obtained as $\sigma_{cr,c} = \sigma_{cr,sl} \frac{b_c}{b_{sl,1}}$, where $\sigma_{cr,c}$ is the relative stress on the compressed edge of the plate, and $b_{sl,1}$ and b_c are geometric values obtained from the stress distribution used for extrapolation (see figure A6.4.2.1).

The relative slenderness of the column $\bar{\lambda}_c$ is defined as follows:

$$\bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr,c}}}$$ for un-stiffened plate

$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,c}}}$$ for stiffened plates

where:

$$\beta_{A,c} = \frac{A_{sl,1,ef}}{A_{sl,1}}$$

$A_{sl,1}$ Is defined above.

$A_{sl,1,ef}$ Area of the effective section due to local plate buckling (see figure A6.4.2.1).

The reduction factor χ_c should be obtained from the indications given in section 35.1.2 of Chapter IX. Un-stiffened plates shall assume a value of the coefficient of imperfection α=0.21, which corresponds to the buckling curve a. For the case of stiffened plates, the value of α be increased to account for the existence of greater initial imperfections in such types of member (stiffening welds). This shall assume the value α_e equal to:

$$\alpha_e = \alpha + \frac{0{,}09}{i/e}$$

where:

α=0,34 (curve b)   for closed section stiffeners.

α=0,49 (curve c)   for open section stiffeners.

$$i = \sqrt{\frac{I_{sl,1}}{A_{sl,1}}}$$

e=max (e_1,e_2)   Largest distance from the centre of gravity of the effective section of the stiffener and of the corresponding part of the support plate (column centre of gravity) to the centre of gravity of the plate or of the gross section of the stiffener alone (or of the set of stiffeners alone, where both sides of the sheet are stiffened) (see figure A6.4.2.1).

A6.4.4 Interactions between plate performance and column buckling

The final reduction factor ρ_c is obtained by interpolation between the coefficients ρ (section A6.4.2) and χ_c (section A6.4.3) as follows:

$$\rho_c = (\rho - \chi_c)\xi(2-\xi) + \chi_c$$

where:

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1 \quad \text{where: } 0 \le \xi \le 1$$

$\sigma_{cr,p}$ critical buckling stress of the plate (section A6.4.2).

$\sigma_{cr,c}$ critical column buckling stress according to the indications given above (section A6.4.3).

ρ   reduction factor due to local buckling of the equivalent orthotropic plate (section A6.4.2).

χ_c Factor reduction due to column buckling (section A6.4.3).

Once ρ_c is known, the area of the effective section of the compressed area of the stiffened panel $A_{c,ef}$ (see section A6.4.1) can then be determined, based on which it is possible to determine the area of the effective section of a generic structural member and its shear modulus.

A6.5 Verification

Member verification for axial compression force and uniaxial bending moment should be performed as follows:

$$\frac{N_{Ed}}{\frac{f_y A_{ef}}{\gamma_{M0}}} + \frac{M_{Ed} + N_{Ed} e_N}{\frac{f_y W_{ef}}{\gamma_{M0}}} \leq 1{,}0$$

where:

A_{ef} Area of the effective reduced section obtained under the action of an axial compression force.

e_N Shift in the position of neutral fibre of the effective reduced section in relation to the neutral fibre of the gross section subject to an axial compression force.

M_{Ed} Design bending moment.

N_{Ed} Design axial force.

W_{ef} Elastic modulus of the effective section subject to a bending moment.

γ_{M0} Partial factor for resistance .

For member subject to axial compression force and biaxial bending, the previous formula appears as follows:

$$\frac{N_{Ed}}{\frac{f_y A_{ef}}{\gamma_{M0}}} + \frac{M_{y,Ed} + N_{Ed} e_{y,N}}{\frac{f_y W_{y,ef}}{\gamma_{M0}}} + \frac{M_{x,Ed} + N_{Ed} e_{z,N}}{\frac{f_y W_{z,ef}}{\gamma_{M0}}} \leq 1{,}0$$

Verification for local sheet buckling is carried out for the resulting values from existent stresses at a distance 0.4a or 0.5b, whichever is smallest, from the end most stressed panel end. In this case, the gross sectional resistance needs to be checked at the end of the panel on the final edge of the panel.

A6.6 'Apparent' axial-longitudinal strain (N-ε_{ap}) diagrams for stiffened compressed sheets

Except in small sections, compressed steel sheets are generally fitted with stiffeners (longitudinal and transverse). This is the case for box sections subject to negative bending stress.

In most cases, due to second-order phenomena, such elements can be susceptible to transverse warping during the elastic and elastic-plastic post-critical phases, and can even produce warping values similar to those of elastic-plastic calculations for failure (see 19.5.1).

The response of compressed stiffened sheets, subject to monotonically increasing axial forces on their plane, adopts a progressively non-linear form with a relatively fragile performance once the load of instability is reached, characterised by a significantly sharp decline in the descending branch of its strength-deformation curve.

The analysis of such elements can be performed using non-linear finite element models of the bottom sheet set and the frame of longitudinal and transverse stiffeners, with any necessary lateral links between this substructure and the rest of the section (webs compressed and flanges). The finite element model should take into consideration equivalent geometric imperfections associated with the forms of critical modes of instability of the stiffened sheet set, as well as its members (sheet panels between stiffeners), representative of the influence on non-linear response of geometric operating

Annex 7: Stiffeners and detailing

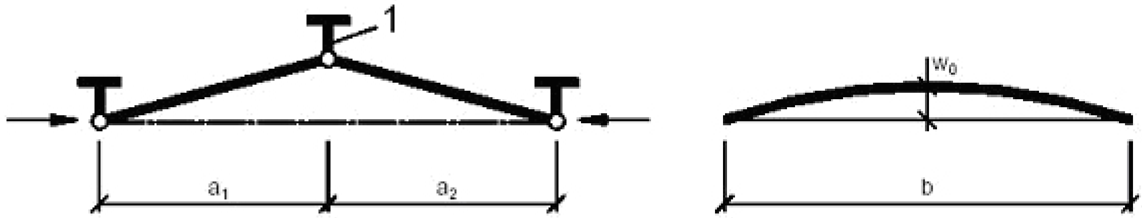
A7.1 General

This Annex lays down criteria for the designing and verification of stiffened elements subject to longitudinal stresses, supplementing the criteria already given for local buckling and concentrated loads (sections 35.4, 35.5, 35.6 and 35.7). It also establishes additional design criteria to those included in section 35.9 and in Annex 6 concerning the checking of longitudinally stiffened sheets subject to longitudinal stresses.

A7.2 Longitudinal stresses

A7.2.1 Minimum requirements for transverse stiffeners

In order to provide a rigid support for a plate s with or without longitudinal stiffeners, intermediate transverse stiffeners should satisfy the criteria given below The transverse stiffener is modelled as a simply supported element with an initial sinusoidal imperfection w_0 of s/300, s being the smallest ofa_1, a_2 or b (see figure A7.2.1), where a_1 and a_2 are the lengths of the panels adjacent to the transverse stiffener in question and b is either the height of the transverse stiffener between the centroids of the flanges or span length of the transverse of the traverse stiffener. Eccentricities should be taken into account.



1. Transverse stiffener

Figure A7.2.1. Transverse stiffener.

The transverse stiffener should carry the deviation forces deriving from the adjacent compressed panels under the assumptions that both adjacent transverse stiffeners are rigid and are kept straight under the combined action of any external load and an axial force, the latter of which is determined according to 35.9.3.3. Compressed plate panels and longitudinal stiffeners are considered to be simply supported at the transverse stiffeners.

Using a second-order elastic method analysis should satisfy the following requirements at the ultimate limit state:

- That the maximum stress in the stiffener should not exceed f_y/γ_{M1}.

- That the additional deflection should not exceed b/300 (see figure A7.2.1).

In the absence of any axial force the transverse stiffener, it can be assumed that both criteria are satisfied provided that the second moment of area of the transverse stiffener I_{st} is not less than:

$$I_{st} = \frac{\sigma_m}{E}\left(\frac{b}{\pi}\right)^4\left(1 + w_0 \frac{300}{b} u\right)$$

where:

$$\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \frac{N_{Ed}}{b}\left(\frac{1}{a_1} + \frac{1}{a_2}\right)$$

$$u = \frac{\pi^2 E e_{max}}{\frac{f_y 300 b}{\gamma_{M1}}} \geq 1{,}0$$

e_{max} The maximum distance from the extreme fibre of the stiffener to the centroid N_{Ed} . The maximum compression force of the adjacent panels to the stiffener, which must not be less than the product of the maximum compression stress multiplied by half the effective compression area of the panel, including stiffeners.

$\sigma_{cr,c}$, $\sigma_{cr,p}$ Defined in Annex 6.

Where the transverse stiffener is subject to an axial compression force, this must be increased by the value $\Delta N_{st}=\sigma_m b_2/ð_2$ to account for deviation forces. The aforementioned requirements remain applicable, but it is not necessary to consider ΔN_{st} when calculating the uniform distribution of stresses on the stiffener, caused by axial force.

In a simplified manner, in the absence of axial force, verification of the aforementioned requirements may be carried out by means of a first-order elastic analysis, considering the additional action of uniformly distributed lateral load acting on the length b of the stiffener and of the following value:

$$q = \frac{\pi}{4}\sigma_m (w_0 + w_{el})$$

where:

σ_m Is defined above in this section.

w_0 Is defined in figure A7.2.1.

w_{el} Elastic deflection, which may be either determined iteratively or be taken as the maximum additional deflection b/300.

Unless an in-depth analysis is carried out, in order to avoid torsional buckling of open cross-section transverse stiffeners, the following criterion should be satisfied:

$$\frac{I_t}{I_p} \geq 5{,}3 \frac{f_y}{E}$$

where:

I_p Polar second moment of area of the stiffener, alone, around the edge fixed to the plate.

I_T St. Venant torsional constant for the stiffener alone.

When warping stiffness is considered, stiffeners should either satisfy the above condition or the criterion $\sigma_{cr} \geq \theta \cdot f_y$

where:

σ_{cr} Elastic critical stress for torsional buckling of the stiffener.
θ Parameter to ensure class 3 performance . A θ value of 6 is recommended.

A7.2.2 Minimum requirements for longitudinal stiffeners

Requirements related to torsional buckling set forth in section A7.2.1 also apply to longitudinal stiffeners.

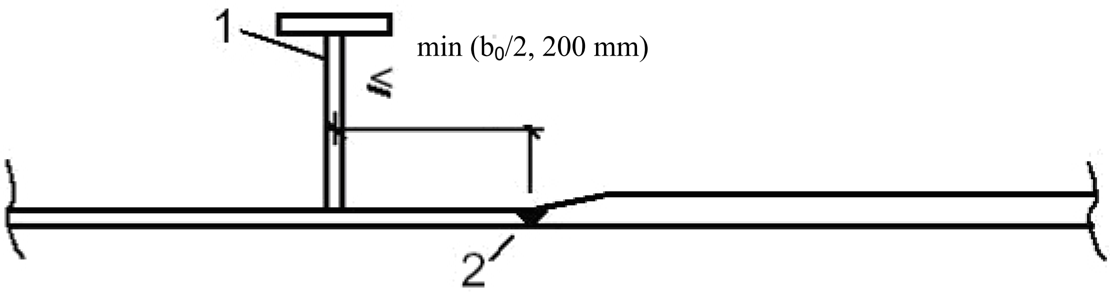
Discontinuous longitudinal stiffeners that do not cross transverse stiffeners through specific openings made in them or which are not connected to both sides of the transverse stiffener:

- Should be used for webs (i.e. they may not be used in flanges).
- not be considered in the global l analysis.
- not be considered in the stress calculation.
- considered in the effective widths of web subpanels.
- considered in the calculations of the elastic critical stresses.

Resistance verification for stiffeners should be performed according to Annex 6

A7.2.3 Welded plates

Plates with changes in plate thickness should be welded in areas close to a transverse stiffener, see figure A7.2.3. The effects of eccentricity do not need to be taken into account, unless the distance to the stiffener from the welded joint exceeds $b_0/2$ or 200 mm, where b_0 is the t width of the plate between longitudinal stiffeners.



1. Transverse stiffener.
2. Transverse weld.

Figure A7.2.3. Welded plates

A7.2.4 Cut-outs in stiffeners

The dimensions of cut-outs in longitudinal stiffeners should be as shown in figure A7.2.4.a.

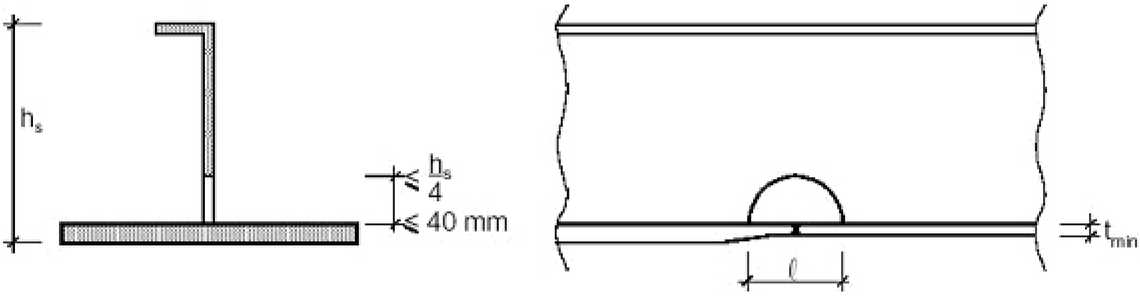


Figure A7.2.4.a. Cut-outs in longitudinal stiffeners

The length □□(see figure A7.2.4.a) should not exceed:

- $\ell \le 6 \cdot t_{min}$ for compressed flat stiffeners
- $\ell \le 8 \cdot t_{min}$ for other compressed stiffeners.
- $\ell \le 15 \cdot t_{min}$ for non-compressed stiffeners.

where t_{min} is the lesser r of the plate thickness.

The limiting values for compressed stiffeners may be increased by $\sqrt{\dfrac{\sigma_{x,Rd}}{\sigma_{x,Ed}}}$ $\sigma_{x,Ed} \le \sigma_{x,Rd}$ when $\ell \le 15 \cdot t_{min}$

The dimensions of cut-outs in transverse stiffeners, should be as shown in figure A7.2.4.b below.

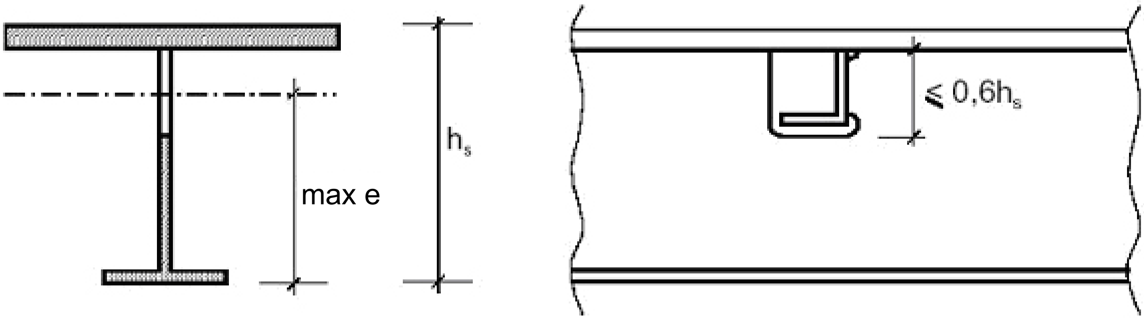


Figure A7.2.4.b. Cut-outs s in transverse stiffeners

The gross web adjacent to the cut-out should resist a shear force of V_{Ed} when :

$$V_{Ed} = \frac{I_{net}}{\max e} \frac{f_{yk}}{\gamma_{M0}} \frac{\pi}{b_G}$$

where:

I_{net} is the second moment of area for the net section of the transverse stiffener.

e Is the maximum distance from n the underside of the flange plate to the neutral fibre of net section (see figure A.7.2.4.b).

b_G Is the length of the transverse stiffener between flanges.

Annex 8: Thermal actions caused by fire

A8.1 General

The process for a structural verification in a fire situation should follow four steps:

1. selection of the fire scenarios appropriate for the design, based on a global strategy in accordance with fire safety standards, for the specific building in the design;

2. determination of the corresponding design fires; this defines the design fire action or, in short, “design fire”, using the curve of temperature increase in gases in the fire enclosure, depending on the time used to characterise the fire action. When selecting the fire design, an appropriate mathematical model from a real fire may be used, or alternatively the standard temperature-time curve that represents the thermal programme for the test furnaces;

3. calculation of the temperature evolution within the structural members as a result of their exposure to fire for the design adopted. If a real fire model is chosen, the calculation shall cover the total duration of the fire, including the cooling phase. If a standard fire is chosen, where there is no cooling phase, the prescribed fire exposure time $t_{fi,requ}$ must be fixed following the specifications of the regulations in force;

4. calculation of the mechanical performance of the structure exposed to fire.

This Annex includes information intended to facilitate the execution of the two first phases mentioned above, since the procedures for the latter two are set out specifically in this Code and are discussed in Chapter XII.

The design fires contained in this Annex correspond to a specific “fire compartment”, defined as an enclosure that is separated from other enclosures that may co-exist in the same building volume using compartment members of a grade and useful surface which is set out in each case by the Technical Code on Building and, where applicable, any relevant Regulations of the Autonomous Governments, local council Orders, etc. Unless otherwise stipulated, a design fire shall be assigned to each situation considered to affect a single fire compartment of the building each time.

The standardised temperature-time curve or, in short, “standard fire”, shall mean the test defined by UNE-EN 1363, and characterised by the variation in temperature over time given in A8.3.1.

A8.2 Verification on structural stability in a fire situation

A8.2.1 Fire resistance required for the structure

In order to adopt the magnitude $t_{fi,nom}$ given below, the regulations in force should be respected.

Unless such regulations specify otherwise, the resistance time required of the structure $t_{fi,requ}$ against a standard fire may be selected using either of the two procedures set out in this Code, which complement each other in the following terms:

a) nominal fire exposure: this is conventionally defined by the duration, expressed in minutes of standard fire, called $t_{fi,nom}$ in this Code, the values of which must be taken from the tables in Section SI 6 of the CTE, the RSCIEI, or, where relevant, the other regulations mentioned in the preceding paragraph;

b) equivalent fire exposure: in this case, a specific analytical criterion is set out to determine the equivalent fire exposure time. This criterion is discussed in A8.5, which defines and quantifies the corresponding representative value of the fire exposure ($t_{e,d}$).

With $t_{fi,requ} = t_{fi,nom}$, or $t_{fi,requ}$ = minimum ($t_{fi,nom}$, $t_{e,d}$) where paragraph b) applies as explained above, the fire safety of the steel structure may then be checked in accordance with the procedures permitted by this Code.

This Annex also includes theoretical-experimental criteria for determining the fire resistance required for structures where, as the other criteria explicitly stated in this Code, the Designer is responsible for selecting the criteria.

A8.2.2 Thermal actions. Net heat flux

Thermal actions on the surface of structural members are defined by the net heat flux $\dot{h}_{net}$ [W/m²] that affects the surface unit of such members that is exposed to fire.

This must be determined using the transmission of heat by convection and radiation:

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \qquad [W/m^2] \qquad (A8.2.1)$$

where:

$\dot{h}_{net,c}$ convection component of thermal flux per surface unit:

$$\dot{h}_{net,c} = \alpha_c \cdot (\Theta_g - \Theta_m) \qquad [W/m^2] \qquad (A8.2.2)$$

$\dot{h}_{net,r}$ radiation component of net heat flux per surface unit:

$$\dot{h}_{net,r} = \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\Theta_r + 273)^4 - (\Theta_m + 273)^4] \qquad [W/m^2] \qquad (A8.2.3)$$

where:

α_c factor for transference of heat by convection [W/m² K]. The α_c factors that apply to the nominal temperature-time curves are given in A8.3 and for natural fire models in A8.6 and A8.7;

Θ_g temperature of gas in the vicinity of the member exposed to fire [ºC], defined by the design fire selected;

Θ_m temperature of the member's surface [ºC]. This is obtained as a result of the thermal analysis of the member in accordance with Chapter XII relating to the structural design in fire situations;

Φ shape factor; if there are no specific data, Φ = 1.0 must be used. A lower value may be used in order to take account of the effects of position and shade;

ε_m emissivity of the member's surface, using $\varepsilon_m = 0.7$;

ε_f emissivity of the flame , generally using $\varepsilon_f = 1.0$;

or the Stephan Boltzmann's constant (= $5.67 \cdot 10^{-8}$ [W/m^2 K^4]);

Θ_r effective fire radiation temperature [ºC]. For members that are completely surrounded by fire, Θ_r may also be included in the gas temperature Θ_g around the member.

For the side of separator members that are not exposed to fire, the net heat flux $\dot{h}_{net}$ must be determined using the equation (A8.2.1) where α_c = 4 [W/m^2 K].

The factor for heat transference by convection must be αc = 9 [W/m^2 K] where it is assumed to include the effects of heat transference by radiation.

A8.2.3 Temperature of gases in the fire compartment. "Design fires"

The temperatures of gases in the fire compartment Θ_g may be used as nominal temperature-time curves in accordance with A8.3, or according to the natural fire models given in A8.6 and A8.7.

In addition to the standard curve in UNE-EN 1363, the external fire curve may also be used between the nominal temperature-time curves for characterise less severe fires that occur in external areas adjacent to the building, or to measure the effects of flames coming out of the windows on external members. It also includes a specific curve for characterising a fire in a pool of hydrocarbons.

The real or natural fire models are those that have varying degrees of complexity and incorporate various physical parameters that are present in the development of a real fire; the simplified calculation models are based on specific physical parameters with limited scope. The advanced models must take account of the gas properties and the exchange of mass and energy.

A8.3 Nominal temperature-time curves

A8.3.1 Standard curve

The standard temperature-time curve is given by:

$$\Theta_g = + 345 \log^{10} (8t + 1) + 20 \qquad [ºC] \qquad (A8.3.1)$$

where:

Θ_g the gas temperature in the fire compartment [ºC];

t *duration of the thermal exposure* [min]

The factor for heat transference by convection is: $\alpha_c = 25$ W/m^2 K.

A8.3.2 External fire curve

The external fire curve is given by:

$$\Theta_g = 660\ (1 - 0.687\ e^{-0.32\,t} - 0.313\ e^{-3.8\,t}) + 20 \qquad [ºC] \qquad (A8.3.2)$$

where:

Θ_g the gas temperature near the member [ºC]

t time duration [min]

The factor for heat transference by convection is: $\alpha_c = 25$ W/m^2 K.

A8.3.3 Hydrocarbon curve

The temperature-time curve for hydrocarbons is given by:

$$\Theta_g = 1{,}080\ (1 - 0.325\ e^{-0.167\,t} - 0.675\ e^{-2.5\,t}) + 20 \qquad [ºC] \qquad (A8.3.3)$$

where:

Θ_g gas temperature in the fire compartment; [ºC]

t time duration [min]

The factor for heat transference by convection is: $\alpha_c = 50$ W/m^2 K.

A8.4 Fire load

The “fire load” should cover all combustible content of the building and all combustible construction members, including coatings and finishes.

Fire loads may be determined:

a) using a national fire load classification according to the type of activity.

In this case, the following are found and added together:

- the actual fire loads of the activity, as given in the classification;
- the fire loads owing to the building (construction members, coatings and finishes) that are not usually included in the classification and which are determined in accordance with the following paragraphs, as applicable;

b) using an individual evaluation: fire loads and their locations must be evaluated, taking account of the anticipated use, furniture and facilities, changes over time, unfavourable tendencies and possible changes in activity that do not entail the preparation of a new design under the regulations in force.

A8.4.1 Fire load density. Design value

The density of the fire load used in the various fire situations should be a design value based on measurements.

The design value may be determined:

- using a national design fire load density classification according to the type of activity;
- in a simplified way, using the expression in this heading (A8.4.1), which takes account of the probability of activation and of the fire-safety measures using partial safety factors;
- in a simplified way for a particular design, by studying the fire loads for each compartment, represented by their characteristic value, taking account of combustion conditions. The probability of activation, active fire-safety measures, evacuation conditions and performance of fire brigades, as well as all additional safety measures and the possible consequences of the fire, must be included using a general safety strategy.

The factor γ_q given in Table A8.5.c, which takes account of the foreseeable consequences of the fire, may be included directly in the value of the design fire load determined using any of the three methods given above, where the verification procedure involved uses it explicitly, such as in the equivalent standard fire time, the parametric curves and the actual fire curves.

The design value of fire load $q_{f,d}$ is defined by:

$$q_{f,d} = q_{f,k} \cdot m \cdot \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \ [\mathrm{MJ/m^2}] \qquad \text{(A8.4.1)}$$

where:

$q_{f,k}$	characteristic fire load density per floor area unit [MJ/m²] (see A8.4.2);
δ_{q1}	partial factor that takes into consideration the fire activation risk starting due to the size of the compartment (see Table A8.4.1.a);
δ_{q2}	Partial factor that takes into consideration the fire activation risk starting due to the type of occupancy (see Table A8.4.1.a);
$\delta_n = \prod_{i=1}^{3} \delta_{ni}$	factor that takes into consideration the influence of different active fire-safety measures i (sprinklers, detection, automatic alarm transmission to fire brigade). Such active measures are generally imposed for safety considerations (see Table A8.4.2.b);
m	combustion factor that takes combustion properties into consideration and represents the proportion of the combustible that burns in the fire, depending on the activity and the type of fire load. For a mixture of predominantly cellulose materials (wood, paper, fabrics, but not for the storage of cellulose), the combustion factor may be assumed as *m* = 0.8. In other cases of mixed materials, m = 1 may be used. In situations where the same material is stored, such as wood, cotton, rubber, gum, linoleum, plastics and artificial fibres, and in the absence of other data, the most unfavourable value, m = 1.4, may be used (low density of material on a large surface).

Table A8.4.1.a. Partial factors δ_{q1} and δ_{q2}

Floor area of sector A_f [m^2]	**Danger of activation δ_{q1}**
20	1.00
25	1.10
250	1.50
2,500	1.90
5,000	2.00
>10,000	2.13

Danger of activation δ_{q2}	**Types of building occupancy**
0.78	Art gallery, museum, swimming pool
1.00	Home, office, residential, education
1.25	Commercial, garages, hospitals, public audiences
1.25	Sectors with low special risk
1.40	Sectors with medium special risk
1.60	Sectors with high special risk

Table A8.4.1.b. Factors for $\delta_{n,i}$

$\delta_{n,i}$ Depending on active fire- safety measures			
Automatic extinguishing	**Automatic detection**		
Automatic water extinguishing system	**Automatic fire detection and alarm**		**Automatic alarm transmission to fire brigade**
$\delta_{n,1}$	**by heat $\delta_{n,2}$**	**by smoke $\delta_{n,2}$**	**$\delta_{n,3}$**
0.61	0.87		0.87

Applying Table A8.4.1.b requires verification that:

- the requirements of European standards relating to sprinkler, detection and alarm installations are fulfilled;
- any active or passive fire protection systems taken into consideration in the design are adequately maintained.

A8.4.2 Density of characteristic fire load

The density of the characteristic fire load $q_{f,k}$ per unit surface area is defined as:

$$q_{f,k} = Q_{fi,k} / A \qquad [\text{MJ/m}^2] \qquad (A8.4.2)$$

where:

$Q_{fi,k}$ characteristic fire load;

A is either the floor area (A_f) of the fire compartment or reference space, or the total internal area of the envelope (A_t), resulting in $q_{f,k}$ or $q_{t,k}$ respectively.

The characteristic fire load is defined as:

$$Q_{fi,k} = \Sigma\, M_{k,i} \cdot H_{ui} \cdot \Psi_i = \Sigma\, Q_{fi,k,i} \qquad [\text{MJ}] \qquad (A8.4.3)$$

where:

$M_{k,l}$ mass of combustible material [kg];

H_{ui} net calorific value [MJ/kg];

[Ψ_i] optional factor that allows adequately protected fire loads to be reduced in accordance with A8.5.3.

In order to estimate the quantity of combustible material, a distinction should be made between:

- permanent fire loads that are not expected to vary throughout the working lifet of a structure, and which must be considered as equal to the values obtained from a subsequent sample, if feasible, or, failing that, they must be considered as equal to nominal values based on the construction members in the design;

- variable fire loads that are likely to change during the working life of the structure, and for the values of which statistical estimates must be used. These values must not be exceeded 80 % of the time.

A8.4.3 Protected fire loads

It is not necessary to consider fire loads located within containers that are designed to resist exposure to fire.

Fire loads located within non-combustible containers that are not specifically designed to resist exposure to fire but remain intact during the fire may be taken into consideration as follows:

- for a fire load at least equal to or greater than 10 % of all protected fire loads, a factor of $\Psi_i = 1.0$ shall apply;

- if this fire load plus unprotected fire loads are not sufficient to heat the remaining fire loads above their ignition temperatures, then a factor of $\Psi_i = 0$ may be applied to the other protected fire loads;
- in other cases, the value of the factor Ψ_i must be determined individually.

A8.4.4 Net calorific values

The net calorific value of the materials should be determined in accordance with UNE-EN ISO 1716:2002.

The moisture content of the materials may be taken into consideration as follows:

$$H_u = H_{u0} (1 - 0.01\, u) - 0.025\, u \qquad \text{[MJ/kg]} \qquad \text{(A8.4.4)}$$

where:

u moisture, as a percentage of weight in dry state;

H_{u0} net calorific value of the dry material.

Table A8.4.4 includes the net calorific value of certain solids, liquids and gases.

Table A8.4.4 – Recommended net calorific value H_u [MJ/kg] of combustible materials for calculations fire loads

Solids	
Wood	17.5
Other cellulosic materials * Fabric * Cork * Cotton * Paper, cardboard * Silk * Straw * Wool	20
Carbon * Anthracite * Coal * Charcoal	30
Chemical products	
Paraffin series * Methane * Ethane * Propane * Butane	50
Olefin series * Ethylene * Propylene * Butane	45
Aromatic series * Benzene * Toluene	40
Alcohols * Methanol * Ethanol * Ethyl alcohol	30
Combustibles * Gasoline, petroleum * Diesel	45
Plastic hydrocarbons * Polyethylene * Polystyrene * Polypropylene	40
Other products	
ABS (plastic)	35
Polyester (plastic)	30
Polyisocyanurate and polyurethane (plastic)	25
Polyvinyl chloride, PVC (plastic)	20
Bitumen, asphalt	40
Leather	20
Linoleum	20
Tyre	30
NOTE: The values in this table do not apply when calculating the energy content of combustibles.	

A8.4.5 Classification of fire loads density according to activity type

The densities of variable fire loads for various buildings, which are classified according to their activity in accordance with Table A8.4.5, refer to the floor area in each sector and should be used as characteristic fire load densities $q_{f,k}$ [MJ/m^2].

Table A8.4.5 – Densities for fire load $q_{f,k}$ [MJ/m^2] for different occupancies activities

Activity	80 % fractile
Home	650
Hospital (living quarters)	280
Hotel (living quarters)	280
Library	1 824
Office	520
School hall	350
Shopping centre	730
Theatre (cinema)	365
Transport (public space)	122
Garage	280

Fire loads from Table A8.4.5 are valid for normal enclosures for the activities shown; i.e. areas where there is no accumulation of fire loads greater than those for the intended use where there is considered to be a mixture of predominantly cellulose materials (given with a combustion factor of m = 0.8). Enclosures dedicated to other activities that do not appear in the table, such as special areas for storage, industrial activities, etc. are considered in accordance with A8.4.2.

Fire loads caused by the building (construction members, coatings and finishes) must be determined in accordance with A8.4.2 and must be added to the fire load densities in Table A8.4.5.

On the other hand, mean fire load values for various activities and commercial and storage uses may be found in the RSCIEI. Multiplying these mean values by 1.6 gives the characteristic fire load value $q_{f,k}$ for inclusion in the calculations referred to in this Annex.

A8.4.6 Variation curves for the rate of heat release in a fire

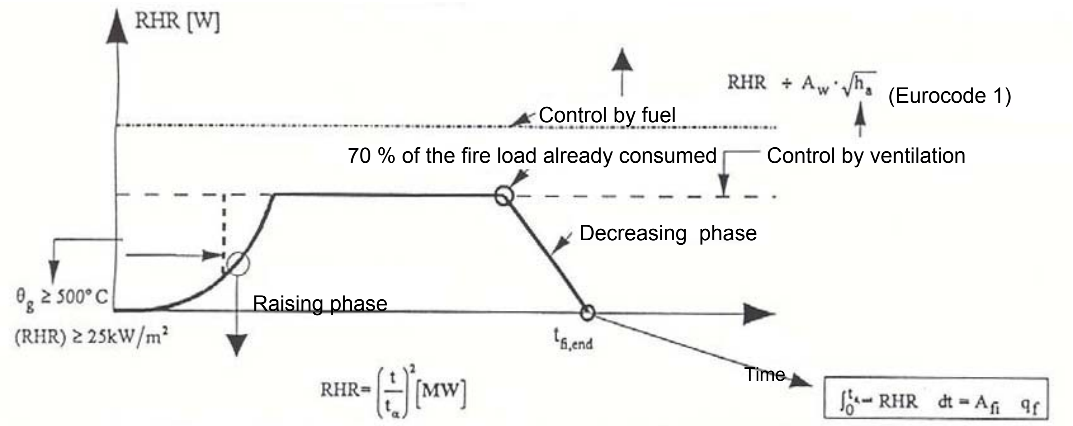


Figure A8.4.6. Design RHR curve of rate of heat generation. The abscissas represent the time t in seconds and the ordinates represent the rate of heat Q (also called RHR) in W (J/s).

The curve in the Figure, which represents the rate of heat generation in a fire over time, distinguishes between three phases: growth, steady and cooling.

The rate of heat release (Q in W) in the raising phase may be defined by the expression:

$$Q = 10^6 \left(\frac{t}{t_\alpha} \right)^2 \qquad \text{(A8.4.6)}$$

where:

t time duration in [s];

t_α is considered to be constant for each case, and represents the time needed to achieve heat release rate of 1 MW. Its value for fires in sectors dedicated to different activities appears in Table A8.4.6.

The curve for the raising phase is limited by a horizontal line corresponding to the plateau phase, with a maximum value of Q, given by:
$Q_{max} = RHR_f A_{fi}$

where:

A_{fi} area of the fire compartment [m^2] if the fire load is evenly distributed, although it may be smaller in the case of a localised fire;

RHR_f maximum rate of heat generation in 1 m^2 of fire, in the case of a fire controlled by the fuel [kW/m^2] (see Table A8.4.6).

The horizontal line, which represents the plateau state, is limited by the decreasing phase. It may be assumed that the latter is a linear decrease that starts when 70 % of the fire load has been consumed and finishes when the fire load has been completely consumed.

If the fire is controlled by ventilation, the level of the horizontal line for the plateau state must be reduced depending on the quantity of oxygen available, using a computer program based on a model for an area, or using the following approximate expression:

$$Q_{max} = 0,10 \cdot m \cdot H_u \cdot A_v \cdot \sqrt{h_{eq}}$$

0,10 = 0.10 [MW] (A8.4.7)
where:

A_v area of the openings [m^2];

h_{eq} mean height of the openings [m];

H_u net calorific value of the wood, equal to 17.5 MJ/kg;

m Combustion factor, taken as being equal to 0.8.

Where the maximum value for heat generation speed is reduced according to the above formula, similarly to a fire controlled by ventilation, the plateau phase must be extended so as to correspond to the energy available, given by the fire load.

Table A8.4.6 – Area of fire progress and RHR_f for different activities

Maximum rate of heat generation RHR_f			
Activity	Rate of fire progress	t_α [s]	RHR_f [kW/m^2]
Home	Medium	300	250
Hospital (living quarters)	Medium	300	250
Hotel (living quarters)	Medium	300	250
Library	Fast	150	500
Office	Medium	300	250
School hall	Medium	300	250
Shopping centre	Fast	150	250
Theatre (cinema)	Fast	150	500
Transport (public space)	Slow	600	250

The values of the factor for rate of fire progress and RHR_f given in Table A8.4.6 are valid when the factor δ_{q2} is equal to 1.0 (see Table A8.4.1.a).

For a very fast-spreading fire, t_α corresponds to the value of 75s.

A8.5 Equivalent fire exposure time

The formula given under this heading depends on the structural material. It does not apply to composite steel and concrete structures, or to wooden structures. It may be used where the design of the members is based on the standard fire exposure.

The equivalent standard fire exposure time is defined by:

$$t_{e,d} = (q_{f,d} \cdot k_b \cdot w_f)\, k_c \qquad \text{[min]} \qquad \text{(A8.5.1)}$$

where:

q_{fd} density of design fire load according to A8.4 [MJ/m^2];

k_b conversion factor relating to the thermal properties of the materials that form the boundary of the fire enclosure;

w_f ventilation factor defined below (non-dimensional);

k_c correction factor depending on the material that makes up the structure, according to Table A8.5.a.

Table A8.5.a – correction factor k_c corresponding to various structural materials (O is the factor for openings defined in A8.6.1).

Section material	Correction factor k_c
Reinforced concrete	1.0
Protected steel	1.0
Unprotected steel	13.7 O

Where no detailed evaluation is performed on the thermal properties of the contour, the conversion factor kb may be taken as:

$k_b = 0.07$ $[\text{min} \cdot m^2/MJ]$ (A8.5.2)

To give greater precision, k_b may be fixed depending on the thermal properties of the contour $b = \sqrt{(\rho\ c\ \lambda)}$ according to Table A8.5.b.

In order to determine b and in the case of materials arranged into various layers, or for walls, floors or ceilings made of different materials, see Table A8.5.b.

Table A8.5.b. Conversion factor k_b depending on the thermal properties of the boundary

$b = \sqrt{(\rho\ c\ \lambda)}$ $[J/m^2 s^{1/2} K]$	k_b $[\text{min} \cdot m^2/MJ]$
b > 2,500	0.04
720 ≤ b ≤ 2,500	0.055
b < 720	0.07

The ventilation factor may be calculated as:

$$w_f = (6.0 / H)^{0.3} [0.62 + 90 (0.4 - \alpha_v)^4 / (1 + b_v \alpha_u)] \geq 0.5 \quad \text{(A8.5.3)}$$

where:

$\alpha_v = A_v/A_f$ relationship between the area of vertical openings in the facade (A_v) and the floor area in the fire compartment (A_f), which must adhere to the limits $0.025 \leq \alpha_v \leq 0.25$;

$\alpha_h = A_h/A_f$ relationship between the area of horizontal openings in the roof (A_h) and the floor area in the fire compartment (A_f);

b_v $= 12.5 (1 + 10\ \alpha_v - \alpha_v^2) \geq 10.0$;

H height of the fire compartment [m].

For small fire compartments (A_f < 100 m^2) without any openings in the roof, the factor w_f may also be calculated as:

$w_f = O^{-1/2} \cdot A_f / A_t$ where:

O factor for openings in accordance with A8.6.1.

The standard fire time to be adopted for the “simplified” verification procedures in Chapter XII of this Code is:

$t = t_{fi,requ} = t_{fi.nom}$, or $t_{fi,requ}$ = minimum ($t_{fi,nom}$, $\gamma_q\ t_{e,d}$) where paragraph b) in A8.2.1 applies, and where:

γ_q partial safety factor that takes account of the foreseeable consequences of the fire in accordance with Table A8.5.c, which, if not included in the design value for the density of the fire load obtained in accordance with (A8.4.1), must be included at this point.

The two formulae for the standard fire safety test discussed in Section 46 of this Code (direct load test or temperature test, under the conditions discussed there) obviously allow for a third alternative formula (equivalent fire time test), as follows:

$t_{fi,requ} < t_{fi,d}$

where:

t_{fid} standard design fire resistance value of members, or duration for the same to reach the resistance limit state for the structural members, $E_{fi,d}/R_{fi,d,t} = 1$, in accordance with Chapter XII of this Code relating to design in a fire situation.

Table A8.5.c Values of γ_q for possible consequences of fire, in accordance with the evacuation height of the building.

Evacuation height	γ_q
Buildings with a downward evacuation height of more than 28 m or ascending evacuation height of more than one storey	2.0
Buildings with a downward evacuation height of between 15 and 28 m or ascending evacuation height up to 2.8 m. Garages under other uses	1.5
Buildings with a downward evacuation height of less than 15 m	1.0

In the case of buildings that are not permitted to remain out of service or where there may be a large number of victims in the event of a fire, such as hospitals, the values given must be multiplied by 1.5.

A8.6 Simplified calculation models

There are two types:

a) “compartment fires”, which are assumed to achieve uniform temperature distribution, depending on time, in the fire compartment. Gas temperatures in the enclosure must be determined depending on appropriate physical parameters, considering at least the density of the fire load and the ventilation conditions. For members inside the compartments, a method for calculating the gas temperature in the enclosure is given in A8.6.1.

For external members, the radiation component for heat flux should be calculated as the sum of total input in the fire compartment and the flames that come out through the openings;

b) “localised fires”, where it is unlikely that there will be sudden, generalised inflammation (flashover) of the materials accumulated in the fire compartment. These fire models assume non-uniform temperature distribution. A8.6.2 gives a method for calculating thermal actions for localised fires. A8.4.6 gives a method for calculating the heat release speed *Q.*

Where these simplified fire models are used, $\alpha_c = 35$ [W/m^2 K] must be used as the factor for heat transference by convection.

A8.6.1 Parametric temperature-time curves

The following temperature-time curves are valid for fire compartments with a constructed surface of not more than 500 m^2, without openings in the cover and with a maximum height of 4 m. It assumes complete combustion of the whole fire load.

If, in applying the procedure set out under this heading, the design value for the density of the fire load $q_{f,d}$ obtained according to (A8.4.1) is applied, and no account has been taken of the consequences of the building's collapse, either in such application or using other procedures, then this value must be multiplied by the partial safety factor γ_q given in Table A8.5.c.

Temperature of gases in the compartment

The temperature-time curves for the heating phase are defined by:

$$\Theta_g = 20 + 1.325\,(1 - 0{,}324\, e^{-0{,}2\, t^*} - 0{,}204\, e^{-1{,}7\, t^*} - 0{,}472\, e^{-19\, t^*})$$

0,324 = 0.324; 0,204 = 0.204, etc. (A8.6.1)

where:

Θ_g Temperature of gases in the fire compartment [ºC]

t^* $= t \cdot \tilde{A}$ [h] (A8.6.2)

with:

t fire time duration [h]

$\tilde{A}$ $= [O/b]^2 / (0.04 / 1{,}160)^2$ [-]

b $= \sqrt{(\rho\ c\ \lambda\ )}$
with the following limits: $100 \leq b \leq 2{,}200$ [J/m^2 s$^{1/2}$ K]

ρ density of construction members that form the internal boundary of the fire sector [kg/m^3];

c specific heat of members of the fire sector envelope [J/kg K];

λ thermal conductivity of the boundary members in the fire sector [W/m K];

O factor for openings: $A_v \sqrt{h_{eq}} / A_t$ [m$^{1/2}$]

with the following limits: $0.02 \leq O \leq 0.20$

A_v total area of vertical openings on all perimeter walls in the sector [m^2];

h_{eq} weighted mean height of openings on all walls [m];

A_t total envelope areas (walls, ceiling and floor, including openings) [m^2].

The values at atmospheric temperature for density ρ, specific heat c and thermal conductivity λ of the corresponding members may be used to calculate the factor b.

If $\tilde{A} = 1$, the equation (A8.6.1) will approximate the standard temperature-time curve.

For an enclosure with various layers of materials, the parameter $b = \sqrt{(\rho\ c\ \lambda\ )}$ must be included as follows:

- if $b_1 < b_2$, $b = b_1$ (A8.6.3)

- if $b_1 > b_2$, a thickness limit of s_{lim} is calculated for the exposed material, in accordance with:

$$s_{\lim} = \sqrt{\frac{3600\ t_{\max}\lambda_1}{c_1\rho_1}}$$ with t_{max} given by the equation (A.6.6.7) [m] (A8.6.4)

if $s_1 > s_{lim}$ then $b = b_1$ (A8.6.4a)

if $s_1 < s_{lim}$ then $$b = \frac{s_1}{s_{\lim}} b_1 + \left(1 - \frac{s_1}{s_{\lim}}\right) b_2$$ (A8.6.4b)

Index 1 is assigned to the layer that is directly exposed to the fire, index 2 to the next layer, etc.

s_i thickness of layer i;

b_i = $\sqrt{(\rho_i c_i \lambda_i)}$

ρ_i density of the same layer;
c_i its specific heat;
λ_i its thermal conductivity.

The average of different factors $b = \sqrt{(\rho\ c\ \lambda\ )}$ of walls, ceilings and floors must be found as follows:

$b = (\Sigma\ (b_j A_j)\ )\ /\ (A_t - A_v)$ (A8.6.5)

where:

A_j area of enclosure member j, excluding openings;

b_j thermal property of member j according to equations (A8.6.3) and (A8.6.4).

Time necessary for gases to reach maximum temperature

The maximum temperature Θ_{max} in the heating phase is achieved for $t^* = t^*_{max}$ given by:

$t^*_{max} = t_{max} \cdot \tilde{A}$ [h] (A8.6.6) with $t_{max} = \max [(0.2 \cdot 10_{-3} \cdot q_{t,d} / O); t_{lim}]$ [h] (A8.6.7)

where:

$q_{t,d}$ design value of the fire load density with reference to the total area A_t of the boundary for the sector $q_{t,d} = q_{f,d} \cdot A_f / A_t$ [MJ/m²]. The following limits must be observed: $50 \leq q_{td} \leq 1{,}000$ [MJ/m²];

$q_{f,d}$ design value of the fire load density with reference to the floor area of the compartment A_f [MJ/m²], multiplied by the factor γ_q if calculated with (A8.4.1);

t_{lim} in [h]. For slow-spreading fires, t_{lim} = 25 min; for medium-spreading fires, t_{lim}= 20 min and for fast-spreading fires, t_{lim} = 15 min. For more details on fire progress area, see A8.4.6.

If t_{lmax} is given by $(0.2 \cdot 10_{-3} \cdot q_{t,d} / O)$, the fire is controlled by ventilation.

Where the fire is controlled by the fuel, and the time t_{max} corresponding to the maximum temperature is given by t_{lim}, $t_{max} = t_{lim}$, the variable t^* in equation (A8.6.1) is replaced by:

$t^* = t \cdot \tilde{A}_{lim}$ [h] (A8.6.2b)

with: $\tilde{A}_{lim} = [O_{lim} / b]^2 / (0.04 / 1{,}160)^2$ (A8.6.8)

where: $O_{lim} = 0.1 \cdot 10^{-3} q_{t,d} / t_{lim}$ (A8.6.9)

If ($O > 0.04$ and $q_{t,d} < 75$ y $b < 1{,}160$), $\tilde{A}_{lim}$ in (A8.6.8) must be multiplied by k, given by:

$$k = 1 + \left(\frac{O - 0{,}04}{0{,}04}\right)\left(\frac{q_{t,d} - 75}{75}\right)\left(\frac{1160 - b}{1160}\right)$$

0,04 = 0.04 (A8.6.10)

Cooling phase

The temperature-time curves for the decreasing phase are given by:

$\Theta_g = \Theta_{max} - 625\,(t^* - t^*_{max} \cdot x)$ for $t^*_{max} \leq 0.5$ (A8.6.11a)

$\Theta_g = \Theta_{max} - 250\,(3 - t^*_{max} \cdot x)\,(t^* - t^*_{max} \cdot x)$ for $0.5 < t^*_{max} < 2$ (A8.6.11b)

$\Theta_g = \Theta_{max} - 250\,(t^* - t^*_{max} \cdot x)$ for $t^*_{max} \geq 2$ (A8.6.11c)

where: t^* is given by (A8.6.2a).

$t^*_{max} = (0.2 \cdot 10_{-3} \cdot q_{t,d} / O) \cdot \tilde{A}$ (A8.6.12)

$x = 1.0$ if $t_{max} > t_{lim}$, or $x = t_{lim} \cdot \tilde{A} / t^*_{max}$ if $t_{max} = t_{lim}$

A8.6.2 Localised fires

The thermal action of a localised fire on a structural member must be calculated using the expression (A8.2.1), based on a form factor *Φ* that takes account of the effects of position and shade. This factor measures the proportion of total heat emitted through radiation by a radiating surface and arriving at a given receiver surface. Where there are no specific data for the calculation, the value of 1, which is the upper limit, may be adopted.

The heat flux of a localised fire on horizontal structural members located above the fire may be determined using the method found under this heading. The given rules are valid if they fulfil the following conditions:

- the diameter of the fire is limited by $D \leq 10$ m;
- the rate of the fire's heat release is limited by $Q \leq 50$ MW.

Two different cases are set out, depending on the relative height of the flames in relation to the ceiling:

a) flames not impacting the ceiling as defined in Figure A8.6.1;

b) flames impacting the ceiling in accordance with Figure A8.6.2.a.

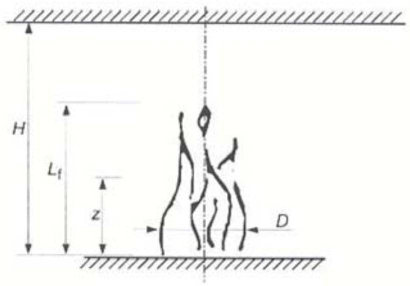


Figure A8.6.2.a.Flames not impacting the ceiling

Flames length L_f in a localised fire is given by:

$$L_f = -1.02\, D + 0.0148\, Q^{2/5} \qquad [\mathrm{m}] \qquad (A8.6.13)$$

where:

D "fire diameter" [m] given in Figure A8.6.1;

Q *rate* of heat release [W] from the fire in accordance with A8.4.6;

H distance [m] between fire source and ceiling.

Case a) (see Figure A8.6.2.a): where the flames do not reach the ceiling of the fire compartment ($L_f < H$), or in the case of an outdoor fire, the following is calculated:

the temperature $\Theta_{(z)}$ of the plume throughout its vertical axis of symmetry, which is given by:

$\Theta_{(z)} = 20 + 0.25\, Q_c^{2/3} (z - z_0)^{-5/3} \leq 900$ [ºC] (A8.6.14)

where:

Q_c convective part of the rate of heat release [W], with by default, $Q_c = 0.8\, Q$;

z height [m] along the flame axis;

z_0 virtual origin of the axis, given by:

$z_0 = - 1.02\, D + 0.00524\, Q^{2/5}$ [m] (A8.6.15)

Flame axis

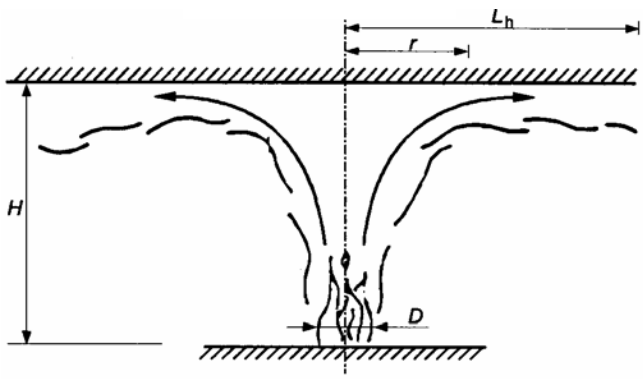


Figure A8.6.2.b. Flames impacting the ceiling

Case b) (see Figure A8.6.2.b): where the flames impacting the ceiling ($L_f \geq H$), the following is calculated:

the heat flux $\dot{h}$ [W/m²] received by radiation in the unit surface area exposed to fire at the level of ceiling, given by the three expressions, depending on the value of the parameter *y:*

$\dot{h}$ = 100,000 if $y \leq 0.30$

$\dot{h}$ = 136,000-121,000 y if $0.30 < y < 1.0$ (A8.6.16)

$\dot{h} = 15{,}000\, y^{-3.7}$ if $y \geq 1.0$

$$y = \frac{r + H + z'}{L_h + H + z'} \quad [-]$$

where:

r horizontal distance [m] between the vertical axis of the fire and the point along the ceiling for which the heat flux is calculated;

H distance [m] between fire source and ceiling;

L_h horizontal length of the flames, given by the following ratio:

$L_h = (2.9\, H\, (Q_H^*)^{0.33}) - H$ [m] (A8.6.17)

Q_H^* non-dimensional rate of heat release, given by:

$Q_H^* = Q / (1.11 \cdot 10^{6} \cdot H^{2.5})$ [-] (A8.6.18)

z' vertical position of the virtual heat source [m] and given by:

$z' = 2.4\ D\ (Q_D^{*\ 2/5} – Q_D^{*\ 2/3})$ when $Q_D^* < 1.0$ (A8.6.19)

$z' = 2.4\ D\ (1.0 - Q_D^{*\ 2/5})$ when $Q_D^* \geq 1.0$

where: $Q_D^* = Q / (1.11 \cdot 10^{6} \cdot D^{2.5})$ [-] (A8.6.20)

Thermal action, represented by the net heat flux $\dot{h}_{net}$ received by the unit surface area of the member exposed to fire at level of the ceiling is given by:

$$\dot{h}_{net} = \dot{h} - \alpha \cdot (\Theta_m - 20) - \Phi \cdot \varepsilon_m \cdot \varepsilon_f \cdot \sigma \cdot [(\Theta_m + 273)^4 - (293)^4]$$ [W/m^2] (A8.6.21)

where the various coefficients depend on the expressions (A8.2.2), (A8.2.3) and (A8.6.16).

In order to obtain the various heat flows $\dot{h}_1$, $\dot{h}_2$, etc. received by the surface unit exposed to fire at the level of the ceiling in the event that there are several separate localised fires, expression (A8.6.16) may be used. The following may be taken for the total heat flux:

$$\dot{h}_{tot} = \dot{h}_1 + \dot{h}_2 \ldots \leq 100.000$$

100.000 = 100 000 [W/m^2] (A8.6.22)

A8.7 Advanced calculation models

Except where more detailed information is available, α_c = 35 [W/m^2 K] should be used as the factor for heat transference by convection.

A8.7.1 One-zone model

A model shall be applied to an area in situations following sudden, generalised inflammation (flashover). The temperature, density, internal energy and air pressure are assumed to be homogeneous in the fire compartment under consideration.

The temperature may be calculated taking into account:

- the solutions to mass and energy conservation equations;
- the exchange of mass between the internal gas, the external air (through the openings) and the fire (pyrolysis speed);
- the exchange of energy between the fire, the internal gas, the walls and the openings.

The ideal gas law to be considered is as follows:

$P_{int} = \rho_g R\ T_g$ [N/m^2] (A8.7.1)

The balance of mass of gases in the compartment is expressed as:

$$\frac{dm}{dt} = \dot{m}_{in} - \dot{m}_{out} + \dot{m}_{fi} \qquad [kg/s] \qquad (A8.7.2)$$

where:

$\frac{dm}{dt}$ loss of mass of gas in the compartment over the unit of time;

$\dot{m}_{out}$ mass of gas exiting through the openings over the unit of time;

$\dot{m}_{in}$ mass of air entering through the openings over the unit of time;

$\dot{m}_{fi}$ mass of products generated by pyrolysis over the unit of time.

The loss of mass of gas and the mass of products generated by pyrolysis may be disregarded. Thus:

$$\dot{m}_{in} = \dot{m}_{out} \qquad (A8.7.3)$$

These mass flows may be calculated using the static pressure owing to the difference in air density at atmospheric temperature and at high temperature.

The energy balance of the gases in the fire compartment may be considered as:

$$\frac{dE_g}{dt} = Q - Q_{out} + Q_{in} - Q_{wall} - Q_{rad} \qquad [W] \qquad (A8.7.4)$$

where:

E_g	internal energy of the gas;	[J]
Q	*rate* of heat release from the fire;	[W]

Q_{out} $= \dot{m}_{out}\, c\, T_f$

Q_{in} $= \dot{m}_{in}\, c\, T_{amb}$

Q_{wall} $= (A_t - A_{h,v})\, \dot{h}_{net}$, loss of energy by boundary surfaces;

Q_{rad} $= A_{h,v}\ or\ T_f^4$, loss of energy by radiation through the openings;

where:

c specific heat [J/kgK];

$\dot{h}_{net}$ given by the expression (A8.2.1);

$\dot{m}$ rate of variation in gas mass [kg/s];

T Temperature [K].

A8.7.2 Two-zone models

A two-zone model is based on a hypothesis according to which the combustion products accumulate in a layer below the ceiling, with a horizontal surface of separation. There are different areas: the top layer, the bottom layer, the fire and its plume, the external air and the walls.

In the top layer, the gas properties may be assumed to be uniform.

The exchange of mass, energy and chemical substance may be calculated between these surfaces.

In a given compartment with an uniformly distributed fire load, a two zone model may be converted into a one-zone model, where there is one of the following situations:

- if the gas temperature in the top layer exceeds 500 ºC;

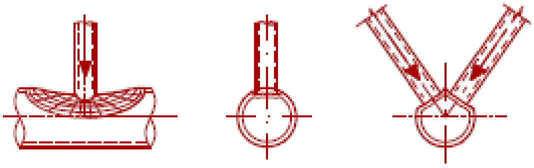
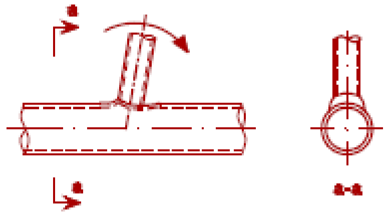
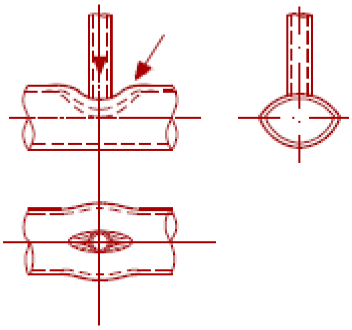
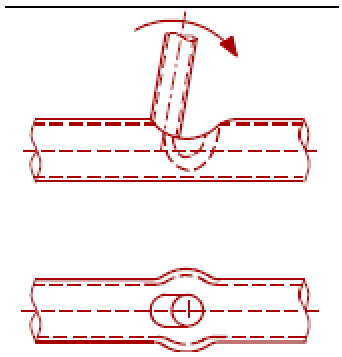
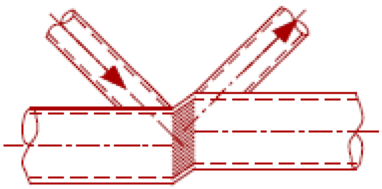
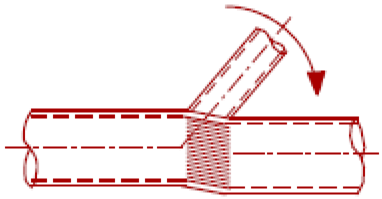
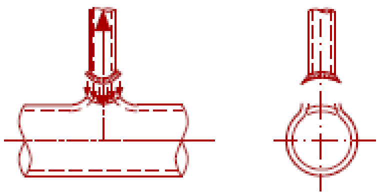
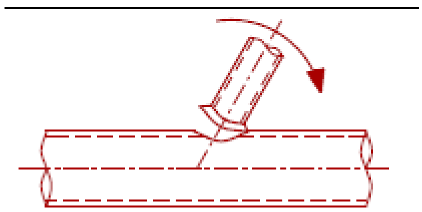
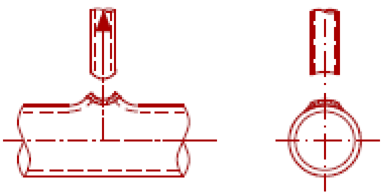
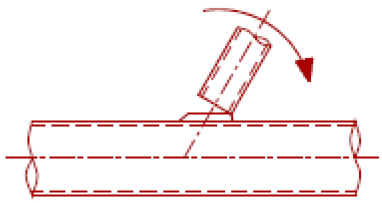
- if the height of the top layer increases to exceed 80 % of the compartment height.

A8.7.3 Computer models for fluid dynamics

A computer model for fluid dynamics may be used to solve the equations digitally in partial derivatives, given the thermodynamic and aerodynamic variables for all points of the compartment. These equations are the mathematical expression of the conservation laws of physics:

- fluid mass is conserved;
- the variation in the quantity of movement of each fluid particle is equal to the sum of the forces exercised on it (Newton's second law);
- the variation in energy is equal to the sum of the heating rate and the work rate on each fluid particle (first law of thermodynamics).

Annex 9: Direct joints for hollow sections

Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		

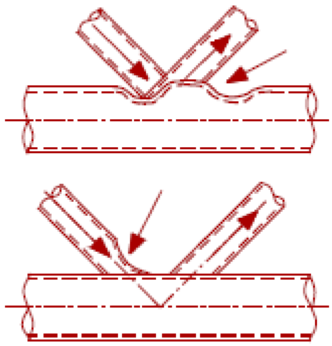
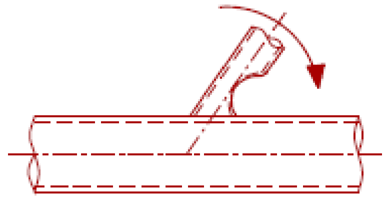
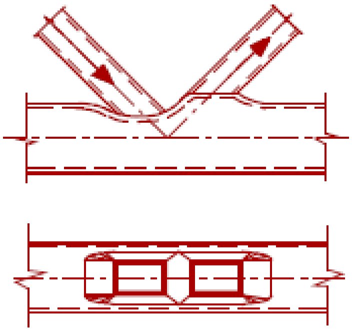
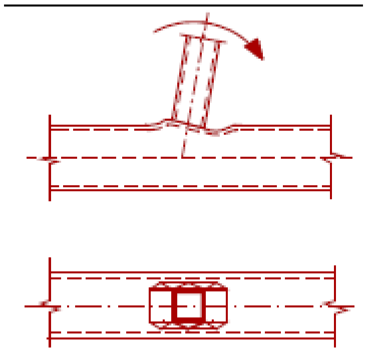
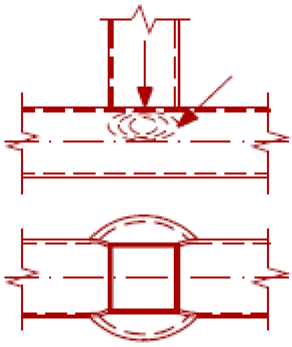
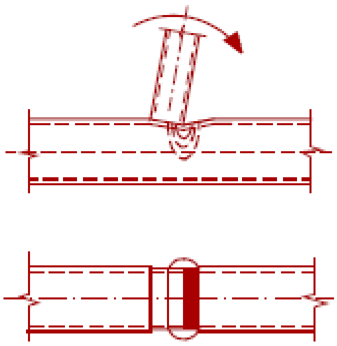
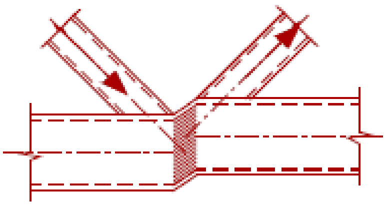
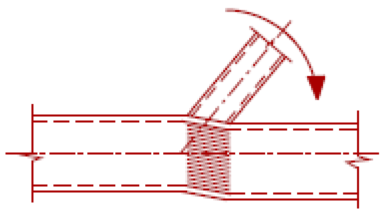
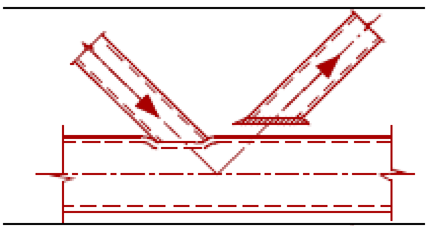
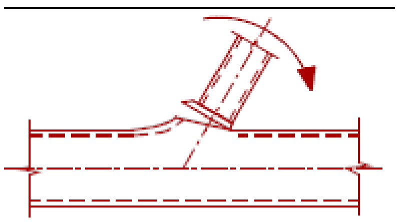
f		

Figure A-9-1: Failure modes for joints between CHS members

Mode	Axial loading	Bending moment
a		
b		
c		
d		

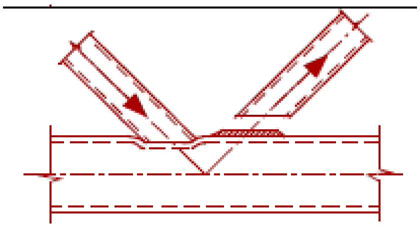
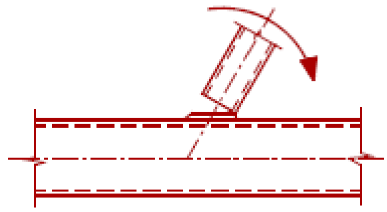
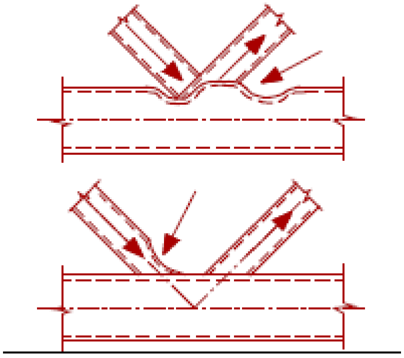
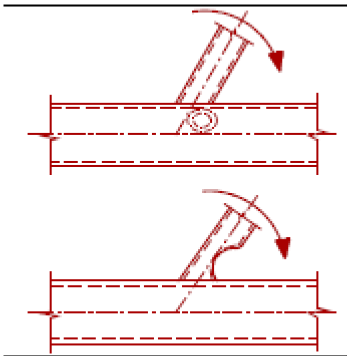
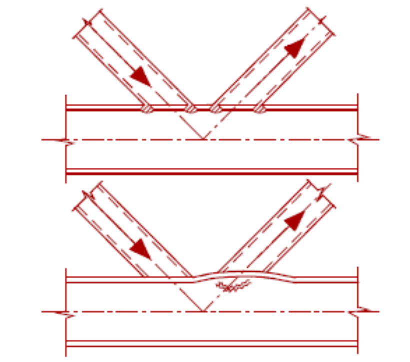
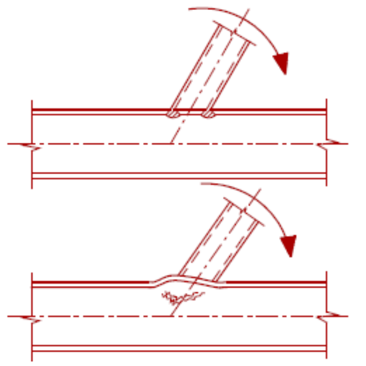
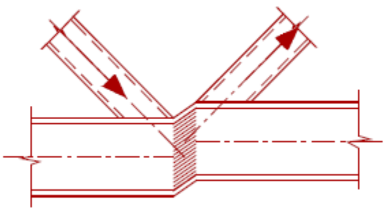
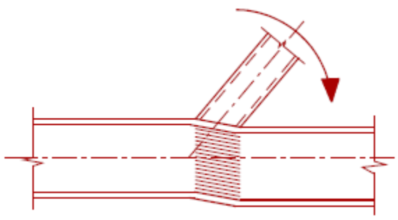
e		
f		

Figure A-9-2: Failure modes for joints between RHS brace member and RHS chord members

Mode	Axial	Bending moment
a	-	-
b		
c		

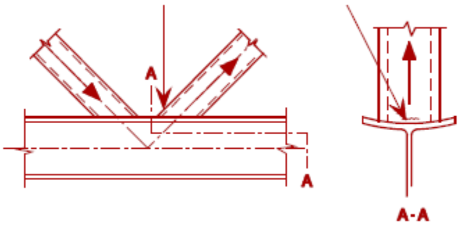
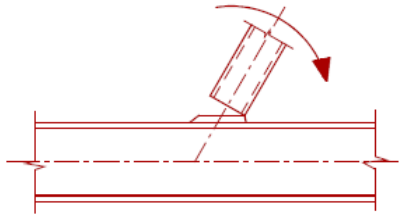
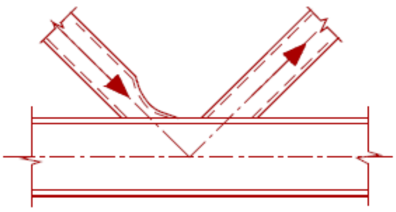
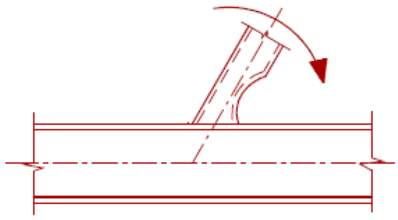
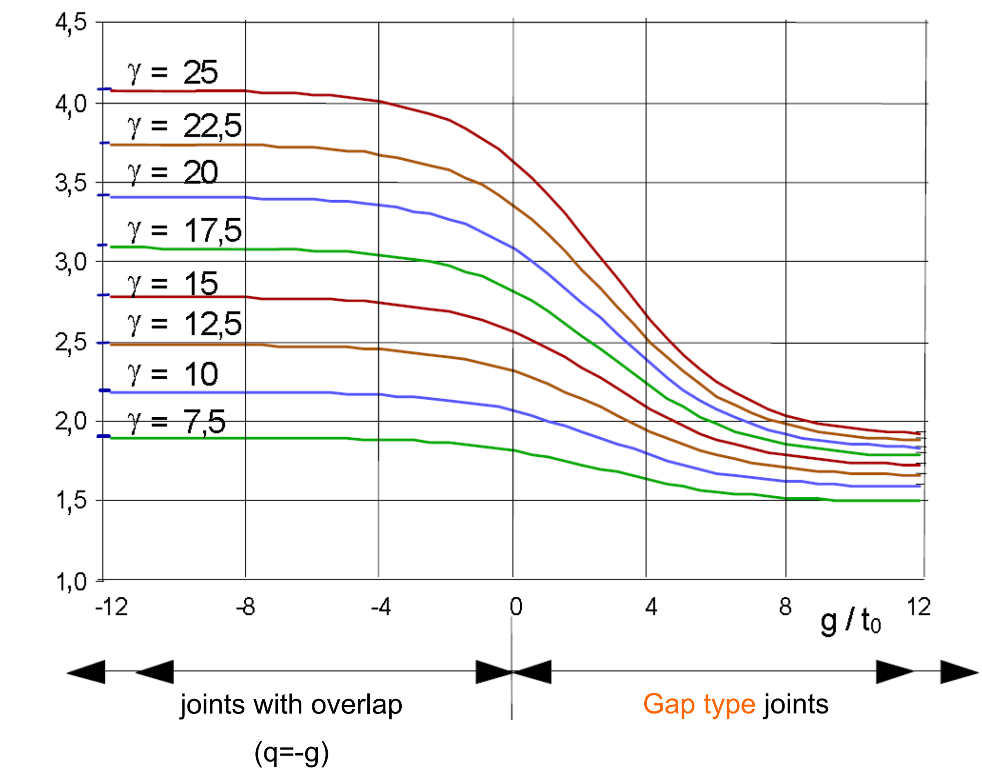
d	-	-
e		
f		

Figure A-9-3: Failure modes for joints between CHS or RHS brace members and I or H section chord members



1,0 = 1.0; 1,5 = 1.5, etc.

Figure A-9-4: Values of the factor *k* for use in Table A-9-1

Table A-9-1: Design axial resistance of welded joints between CHS brace members and CHS chords

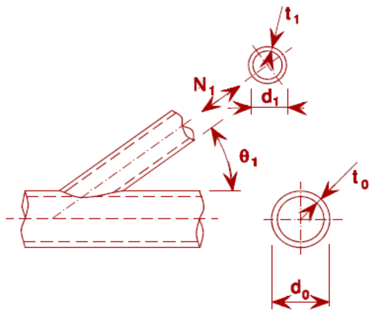
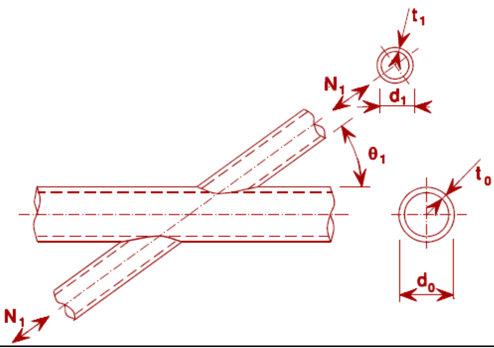
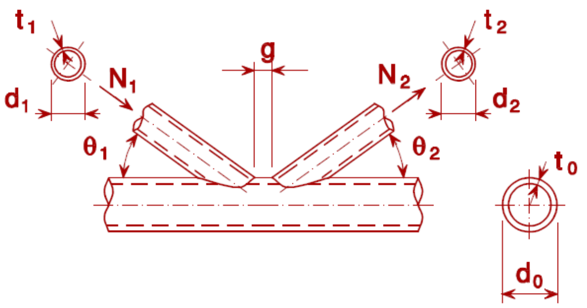
<table>
<tr><td colspan="2">Chord face failure - T and Y joints</td></tr>
<tr><td></td><td>$$N_{1,Rd} = \frac{\gamma^{0.2} k_p f_{y0} t_0^2}{\sin\theta_1}(2.8+14.2\beta^2)/\gamma_{M5}$$</td></tr>
<tr><td colspan="2">Chord face failure - X joints</td></tr>
<tr><td></td><td>$$N_{1,Rd} = \frac{\gamma^{0.2} k_p f_{y0} t_0^2}{\sin\theta_1}\frac{5.2}{(1-0.81\beta)}/\gamma_{M5}$$</td></tr>
<tr><td colspan="2">Chord face failure - K and N gap or overlap joints</td></tr>
<tr><td></td><td>$$N_{1,Rd} = \frac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1.8+10.2\frac{d_1}{d_0}\right)/\gamma_{M5}$$
$$N_{2,Rd} = \frac{\sin\theta_1}{\sin\theta_2}N_{1,Rd}$$</td></tr>
<tr><td colspan="2">Failure by punching - K, N and KT joints , and all T, Y and X joints
[i = 1, 2 or 3]</td></tr>
<tr><td colspan="2">When $d_i \le d_0 - 2t_0$: $N_{i,Rd} = \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_i \frac{1+\sin\theta_i}{2\sin^2\theta_i}/\gamma_{M5}$</td></tr>
<tr><td colspan="2">k_g y k_p factors</td></tr>
<tr><td colspan="2">$k_g = \gamma^{0.2}\left(1+\frac{0.024\gamma^{1,2}}{1+\exp(0.5g/t_0-1.33)}\right)$ (see Figure A-9-4)</td></tr>
<tr><td colspan="2">For $n_p > 0$ (compression): $k_p = 1 - 0.3\, n_p(1+n_p)$ but $k_p \le 1.0$
For $n_p > 0$ (tension): $k_p = 1.0$</td></tr>
</table>

Table A-9-2: Design axial resistance of welded joints connecting gusset plates to CHS members

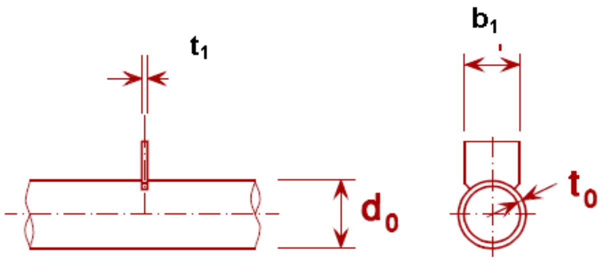
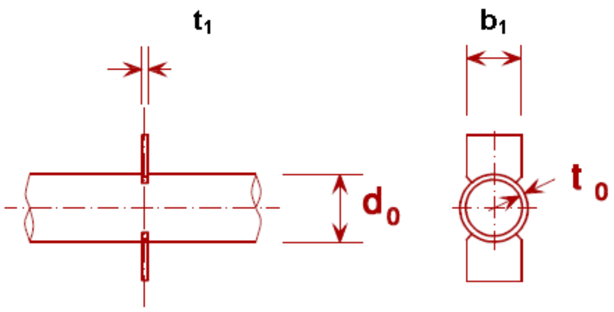
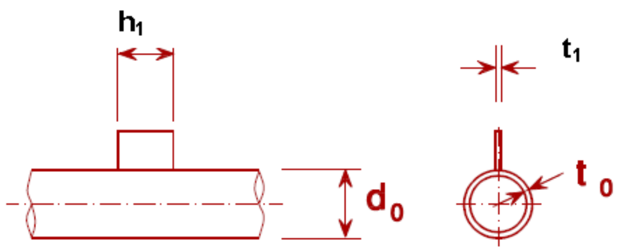
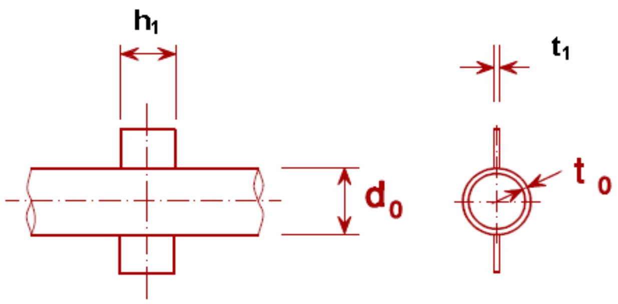
<table>
<tr><td colspan="2">Chord face failure</td></tr>
<tr><td>
</td><td>$N_{1,Rd} = k_p f_{y0} t_0^2 (4+20\beta^2)/\gamma_{M5}$
$M_{ip,1,Rd} = 0$
$M_{op,1,Rd} = 0.5\, b_1 N_{1,Rd}$</td></tr>
<tr><td>
</td><td>$N_{1,rd} = \dfrac{5k_p f_{y0} t_0^2}{1-0.81\beta} / \gamma_{M5}$
$M_{ip,1,Rd} = 0$
$M_{op,1,Rd} = 0.5\, b_1 N_{1,Rd}$</td></tr>
<tr><td>
</td><td>$N_{1,Rd} = 5k_p f_{y0}\, t_0^2\, (1+0.25\eta)/\gamma_{M5}$
$M_{ip,1,Rd} = h_1 N_{1,Rd}$
$M_{op,1,Rd} = 0$</td></tr>
<tr><td>
</td><td>$N_{1,Rd} = 5k_p f_{y0}\, t_0^2\, (1+0.25\eta)/\gamma_{M5}$
$M_{ip,1,Rd} = h_1 N_{1,Rd}$
$M_{op,1,Rd} = 0$</td></tr>
<tr><td colspan="2">punching shear failure</td></tr>
<tr><td colspan="2">$\sigma_{max} t_1 = (N_{Ed,1} / A_1 + M_{Ed,1}/W_{el,1}) t_1 \le 2 t_0 (f_{y0} / \sqrt{3} / \gamma_{M5}$</td></tr>
<tr><td>Validity range</td><td>Factor k_p</td></tr>
<tr><td>In addition to the limits given in Table 64.6.1:
$\beta \ge 0.4$ y $\eta \le 4$
where $\beta = b1 / d0$ and $\eta = h1 / d0$</td><td>For $n_p > 0$ (compression):
$k_p = 1 - 0.3\, n_p\, (1 + n_p)$ but $k_p \le 1.0$
For $n_p \le 0$ (tension): $k_p = 1.0$</td></tr>
</table>

Table A-9-3: Design resistance of welded joints connecting I, H or RHS sections CHS members

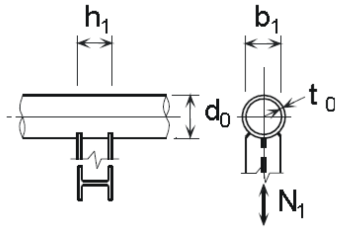
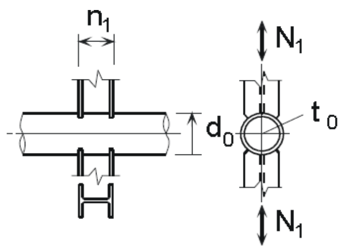
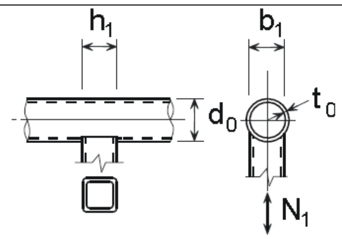
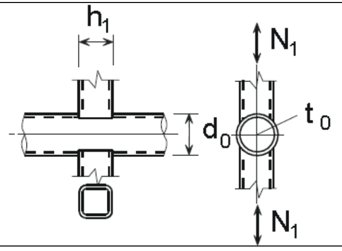
Chord face failure	
	$N_{1,Rd} = k_p f_{y0} t_0^2 (4+20\beta^2)(1+0.25\eta)/\gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}/(1+0.25\eta)$ $M_{op,1,Rd} = 0.5\ b_1 N_{1,Rd}$
	$N_{1,rd} = \dfrac{5k_p f_{y0} t_0^2}{1-0.81\beta}(1+0.25\eta)/\gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}/(1+0.25\eta)$ $M_{op,1,Rd} = 0.5\ b_1 N_{1,Rd}$
	$N_{1,Rd} = k_p f_{y0}\ t_0^2\ (4+20\beta^2)(1+0.25\eta)/\gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}$ $M_{op,1,Rd} = 0.5\ b_1 N_{1,Rd}$
	$N_{1,rd} = \dfrac{5k_p f_{y0} t_0^2}{1-0.81\beta}(1+0.25\eta)/\gamma_{M5}$ $M_{ip,1,Rd} = h_1 N_{1,Rd}$ $M_{op,1,Rd} = 0.5\ b_1 N_{1,Rd}$
Shear failure by punching	
I or H sections with $\eta > 2$ (for axial compression load and moments outside the plane) and RHS sections: $\sigma_{max} t_1 = (N_{Ed,1} / A_1 + M_{Ed,1}/W_{el,1}) t_1 \leq t_0 (f_{y0} / \sqrt{3} / \gamma_{M5}$ $\sigma_{max} t_1 = (N_{Ed,1} / A_1 + M_{Ed,1}/W_{el,1}) t_1 \leq 2t_0 (f_{y0} / \sqrt{3} / \gamma_{M5}$ where t1 is the thickness of the flange on the transverse I or H beam, or of the hollow section.	
Validity range	Factor k_p
In addition to the limits given in Table 64.6.1: $\beta \geq 0.4$ y $\eta \leq 4$ where $\beta = b1 / d0$ and $\eta = h1 / d0$	For $n_p > 0$ (compression): $k_p = 1 - 0.3\ n_p\ (1 + n_p)$ but $k_p \leq 1.0$ For $n_p \leq 0$ (traction): $k_p = 1.0$

Table A-9-4: Design resistance moments of welded joints between CHS members and CHS chords

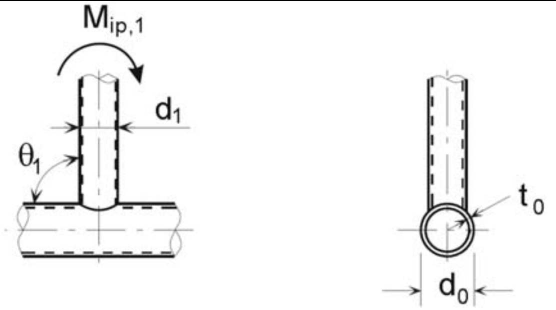
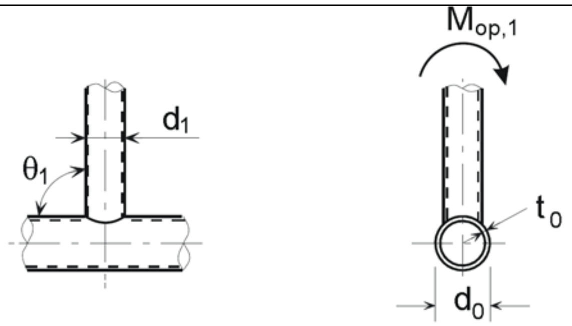
Chord face failure - T, X and Y joints	
	$M_{ip,1,Rd} = 4.85 \dfrac{f_{y0} t_0^2 d_1}{\sin\theta_1} \sqrt{\gamma} \beta k_p / \gamma_{M5}$
Chord face failure - K, N, T, X and Y joints	
	$M_{op,1,Rd} = \dfrac{f_{y0} t_0^2 d_1}{\sin\theta_1} \dfrac{2.7}{1 - 0.81\beta} k_p / \gamma_{M5}$
Punching shear failure – K and N gap joints , and T, X and Y joints	
Where $d_1 \leq d_0 - 2t_0$: $M_{ip,1,Rd} = \dfrac{f_{y0} t_0 d_1^2}{\sqrt{3}} \dfrac{1 + 3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$ $M_{op,1,Rd} = \dfrac{f_{y0} t_0 d_1^2}{\sqrt{3}} \dfrac{3 + \sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$	
k_p factor	
For $n_p > 0$ (compression): $k_p = 1 - 0.3\, n_p\, (1 + n_p)$ but $k_p \leq 1.0$ For $n_p \leq 0$ (tension : $k_p = 1.0$	

Table A-9-5: Design criteria for special types of welded joints between CHS and CHS

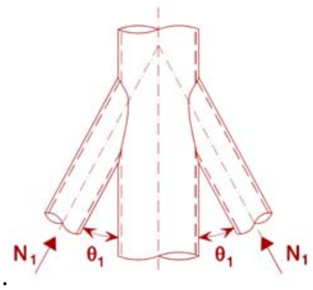
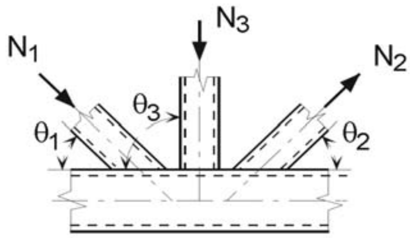
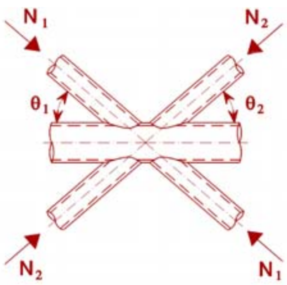
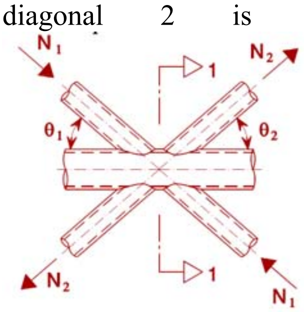
<table>
<tr><th>Joint type</th><th>Design criteria</th></tr>
<tr><td>Forces may be either tension or compression, but should act in the same direction both members

</td><td>$N_{1,Ed} \leq N_{1,Rd}$
where $N_{1,Rd}$ is the value of $N_{1,Rd}$ for an X joint from in h Table A-9-1.</td></tr>
<tr><td>Members 1 and 3 here are in compression, and diagonal 2 here is in tension .

</td><td>$N_{1,Ed} \sin \theta_1 + N_{3,Ed} \sin \theta_3 \leq N_{1,Rd} \sin \theta_1$
$N_{2,Ed} \sin \theta_2 \leq N_{1,Rd} \sin \theta_1$
where $N_{1,Rd}$ is the value of $N_{1,Rd}$ for a K joint from ith Table A-9-1 but with $\frac{d_1}{d_0}$ replaced by: $\frac{d_1 + d_2 + d_3}{3d_0}$</td></tr>
<tr><td>All the diagonals must always be in either compression or traction.

</td><td>$N_{1,Ed} \sin \theta_1 + N_{2,Ed} \sin \theta_2 \leq N_{x,Rd} \sin \theta_x$
where $N_{x,Rd}$ is the value of $N_{x,Rd}$ for an X joint from Table A-9-1, where $N_{x,Rd} \sin \theta_x$ is the :
$\left| N_{1,Rd} \sin \theta_1 \right|$ and $\left| N_{2,Rd} \sin \theta_2 \right|$</td></tr>
<tr><td>Diagonal 1 is always in compression, and diagonal 2 is always in tension

</td><td>$N_{i,Ed} \leq N_{i,Rd}$
where $N_{1,Rd}$ is the value of $N_{1,Rd}$ for a K joint in accordance with Table A-9-1. In gap types at section 1-1the chord also fulfil:
$$\left[\frac{N_{0,Ed}}{N_{pl,0,Rd}}\right]^2 + \left[\frac{V_{0,Ed}}{V_{pl,0,Rd}}\right]^2 \leq 1.0$$</td></tr>
</table>

Table A-9-6: Reduction factors for joints

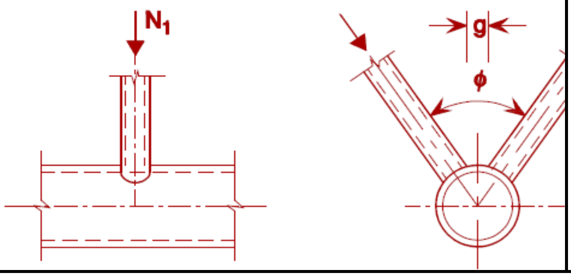
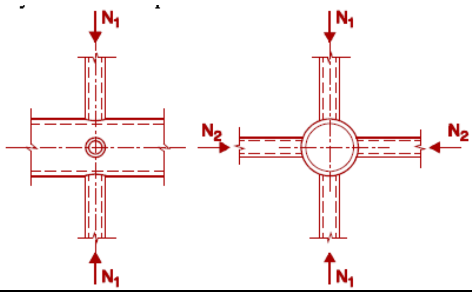
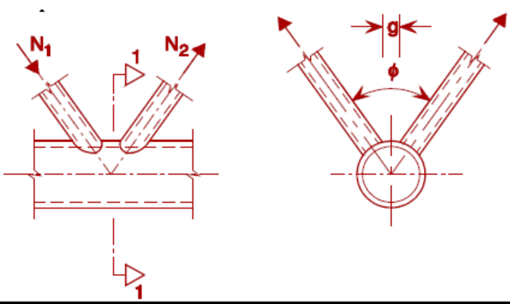
Joint type	Reduction factor μ
TT joint	$60° \leq \varphi \leq 90°$
1 Member1 may be in either or compression. 	$\mu = 1.0$
XX joint	
Member 1 and 2 may be in either tension or compression. $N_{2,Ed}/N_{1,Ed}$ is negative if one member is in tension and the other is in compression. 	$\mu = 1{+}0.33 N_{2,Ed} / N_{1,Ed}$ taking account of the sign of $N_{1,Ed}$ and $N_{2,Ed}$ where: $\lvert N_{2,Ed} \rvert \leq \lvert N_{1,Ed} \rvert$
KK joint	$60° \leq \varphi \leq 90°$
Member 1 is always in compression, and diagonal 2 is always in tension . 	$\mu = 0.9$ Provided that, in a gap s-type joint 1-1 section the chord satisfies : $\left[\frac{N_{0,Ed}}{N_{pl,0,Rd}}\right]^2 + \left[\frac{V_{0,Ed}}{V_{pl,0,Rd}}\right]^2 \leq 1.0$

Table A-9-7: Design axial resistance of welded joints between square or circular hollow sections

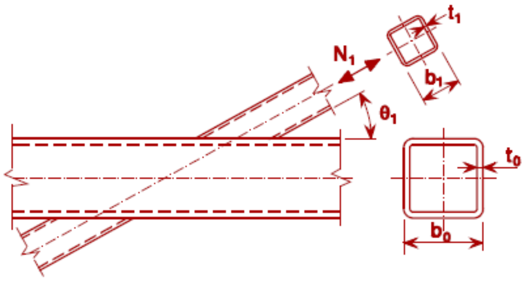
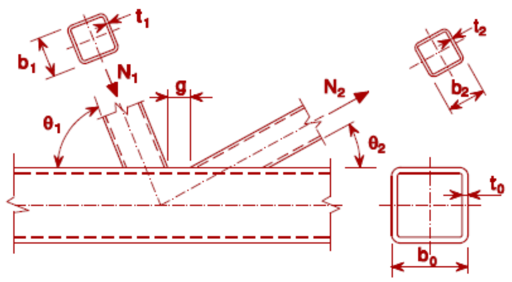
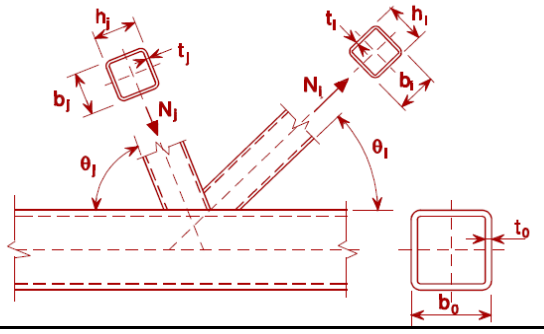
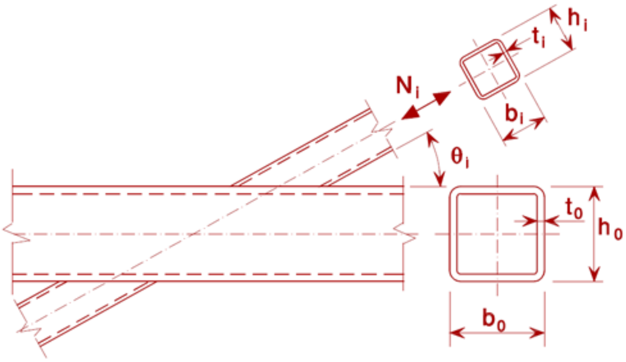
Joint type	Design resistance [i = 1 or 2, j = overlapped brace member]
T, Y and X joints	Chord face failure $\beta \le 0.85$
	$N_{1,Rd} = \frac{k_n f_y t_0^2}{(1-\beta)\sin\theta_1}\left(\frac{2\beta}{\sin\theta_1}+4\sqrt{1-\beta}\right)/\gamma_{M5}$
K and N gap joints	Chord face failure $\beta \le 1.0$
	$N_{1,Rd} = \frac{8.9\gamma^{0.5} k_n f_{y0} t_0^2}{\sin\theta_1}\left(\frac{b_1+b_2}{2b_0}\right)/\gamma_{M5}$
K and N overlap joints *)	Brace failure 25 % $\le \lambda_{ov} <$ 50 %
Diagonal i or diagonal j may be in either tension or compression, but one must be in tension and the other compression. 	$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + 2h_i \frac{\lambda_{ov}}{50} - 4t_i\right)/\gamma_{M5}$ Brace failure 50 % $\le \lambda_{ov} <$ 80 % $N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + 2h_i - 4t_i\right)/\gamma_{M5}$ Brace failure $\lambda_{ov} \ge$ 80 % $N_{i,Rd} = f_{yi} t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i\right)/\gamma_{M5}$
Parameters b_{eff}, $b_{e,ov}$ and k_n	
$b_{eff} = \frac{10}{b_0/t_0}\frac{f_{y0}t_0}{f_{yi}t_i}b_i$ but $b_{eff} \le b_i$	For $n > 0$ (compression): k_n=1.3 - $\frac{0.4n}{\beta}$ but $k_n \le 1.0$ For $n \le 0$ (): k_n = 1.0
$b_{e,ov} = \frac{10}{b_j/t_j}\frac{f_{yj}t_j}{f_{yi}t_i}b_i$ but $b_{e,ov} \le b_i$	
For for circular braces s, the resistances above are multiplied by $\pi/4$, replacing b_1 and h_1 with d_1 and b_2, and h_2 with d_2 . *) only the overlapping member I needs be checked. The brace efficiency (i.e. the design resistance of the joint divided by the plastic design resistance of the brace member) of the overlappedbrace member j should be taken as equal to that of the overlapping brace member. See also Table 64.7.1 in the body of the Code.	

Table A-9-8: Design resistance of welded T, X and Y joints between RHS or CHS braces and RHS

Joint type	Design resistance
	*Chord face failure* $\beta \leq 0.85$
	$N_{1,Rd} = \frac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1}\left(\frac{2\eta}{\sin\theta_1} + 4\sqrt{1-\beta}\right) / \gamma_{M5}$
	Chord side wall Buckling e[1] $\beta = 1.0$[2]
	$N_{1,Rd} = \frac{k_n f_b t_0}{\sin\theta_1}\left(\frac{2h_1}{\sin\theta_1} + 10t_0\right) / \gamma_{M5}$
	Brace failure $\beta \geq 0.85$
	$N_{1,Rd} = f_{y1} t_1 (2h_1 - 4t_1 + 2b_{eff}) / \gamma_{M5}$
	Punching shear $0.85 \leq \beta \leq (1 - 1/\gamma)$
	$N_{1,Rd} = \frac{f_{y0} t_0}{\sqrt{3}\sin\theta_1}\left(\frac{2h_1}{\sin\theta_1} + 2b_{e,p}\right) / \gamma_{M5}$

[1] For X joints with $\cos\theta_1 > h_1/h_0$, the seam's resistance to shearing force is also tested in accordance with Table A-9-9, taking the smaller of this or the above as the design resistance of the joint.

[2] For $0.85 \leq \beta \leq 1.0$ use a linear interpolation between the value for chord face with $\beta = 0.85$ and the governing value for chord side wall at failure $\beta = 1.0$ (side wall buckling or chord shear).

For for circular braces, the resistances above are multiplied by $\pi/4$, replacing b_1 and h_1 with d_1 and b_2, and h_2 with d_2 .

For tension : $f_b = f_{y0}$ For compression: $f_b = \chi f_{y0}$ (T and Y joints) $f_b = 0{,}8\, \chi f_{y0} \sin\theta_1$ (X joints)	$b_{eff} = \frac{10}{b_0/t_0}\frac{f_{y0}t_0}{f_{y1}t_1} b_1$ but $b_{eff} \leq b_1$
where χ is the reduction factor for flexural buckling caused by bending using the corresponding buckling curve and a normalized slenderness λ determined from $\bar{\lambda} = 3.46 \frac{\left(\frac{h_0}{t_0} - 2\right)\sqrt{\frac{1}{\sin\theta_1}}}{\pi\sqrt{\frac{E}{f_{y0}}}}$	$b_{e,p} = \frac{10}{b_0/t_0} b_1$ but $b_{e,p} \leq b_i$ For $n > 0$ (compression): $k_n = 1.3 - \frac{0.4n}{\beta}$ but $k_n \leq 1.0$ For $n \leq 0$ (): $k_n = 1.0$

Table A-9-9: Design resistance of welded K and N joints between RHS or CHS braces and RHS chords

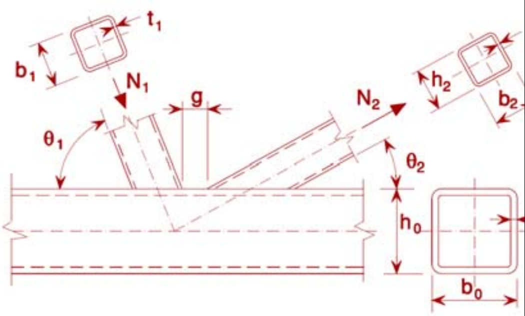
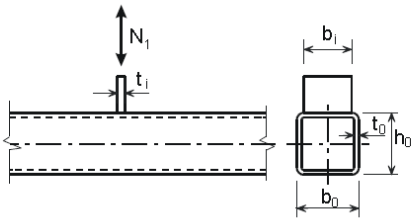
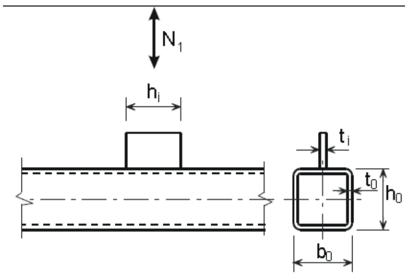
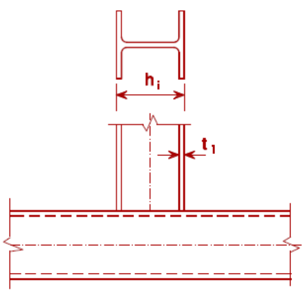
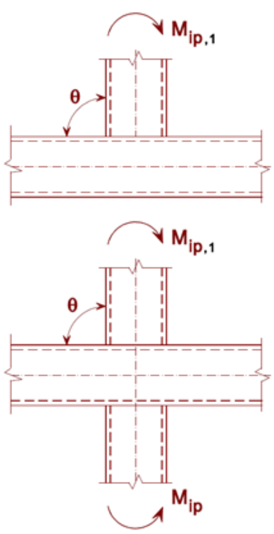
Joint type	Design resistance [i = *1* or *2*]
K and N joints with separation	Chord face failure
	$N_{i,Rd} = \dfrac{8.9 k_n f_{y0} t_0^2}{\sin\theta_i}\left(\dfrac{b_1 + b_2 + h_1 + h_2}{4 b_0}\right) / \gamma_{M5}$
	Brace
	$N_{i,Rd} = \dfrac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{0,Rd} = \left[\left(A_0 - A_v\right) f_{y0} + A_v f_{y0}\sqrt{1 - \left(V_{Ed}/V_{pl,Rd}\right)^2}\right] /$
	Brace failure
	$N_{i,Rd} = f_{yi} t_i (2h_i - 4t_i + b_i + b_{eff}) / \gamma_{M5}$
	Punching shear $\beta \leq (1 - 1/\gamma)$
	$N_{i,Rd} = \dfrac{f_{y0} t_0}{\sqrt{3}\sin\theta_i}\left(\dfrac{2h_i}{\sin\theta_i} + b_i + b_{e,p}\right) / \gamma_{M5}$
K and N joints with overlap	As in Table A-9-7.
For braces for circular sections, the resistances above are multiplied by $\pi/4$, replacing b_1 and h_1 with d_1 and b_2, and h_2 with d_2, except for shearing force of the seam.	
$A_v = (2h_0 + \alpha b_0) t_0$ for a brace for a square or rectangular member : where g is the gap $\alpha = \sqrt{\dfrac{1}{1 + \dfrac{4g^2}{3t_0^2}}}$.	$b_{eff} = \dfrac{10}{b_0 / t_0} \dfrac{f_{y0} t_0}{f_{yi} t_i} b_i$ but $b_{eff} \leq b_i$
For diagonals or uprights for a circular section: $\alpha = 0$	$b_{e,p} = \dfrac{10}{b_0 / t_0} b_1$ but $b_{e.p} \leq b_i$ For $n > 0$ (compression): $k_n = 1.3 - \dfrac{0.4n}{\beta}$ but $k_n \leq 1.0$ For $n \leq 0$ (tension): $k_n = 1.0$

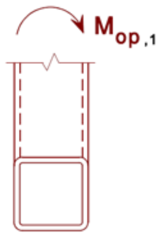
Table A-9-10: Design resistance of welded joints connecting gusset plates K and N joints between sheets or brace members for I or H sections and RHS

Transverse plate	Chord face failure $\beta \le 0.85$
	$$N_{1,Rd} = k_n f_{y0} t_0^2 \frac{2+2.8\beta}{\sqrt{1-0.9\beta}} / \gamma_{M5}$$
	Chord side wall flattening when $b_1 \ge b_0 - 2t_0$
	$N_{1,Rd} = k_n f_{y0} t_0 (2t_1 + 10t_0) / \gamma_{M5}$
	Punching shear when $b_1 \le b_0 - 2t_0$
	$$N_{1,Rd} = \frac{f_{y0} t_0}{\sqrt{3}} (2t_1 + 2b_{e,p}) / \gamma_{M5}$$
Longitudinal plate	Chord face failure
 $t_1/b_0 \le 0.2$	$N_{1,Rd} = k_m f_{y0} t_0^2 \left(2h_1/b_0 + 4\sqrt{1 - t_1/b_0}\right) / \gamma_{M5}$
I or H plate section	
	As a conservative approximation, if $\eta \ge 2\sqrt{1-\beta}$, for an I or H section, $N_{1,Rd}$ may be assumed to be equal to the the design resistances of two transverse plates of similar dimensions to the flanges on the I or H section, determined as specified above. If $\eta < 2\sqrt{1-\beta}$, a linear interpolation betweenone and two plates should be made the heet ($\eta=0$) ($\eta=2\sqrt{1-\beta}$). $M_{ip,1,Rd} = N_{1,Rd}(h_1 - t_1)$ where $N_{1,Rd}$ is the resistance of one of the flanges and β is the relationship between the width of the flange on the I or H beam and the width of the RHS seam.

Validity range	
In addition to the limits given in Table 64.7.1: $0.5 \leq \beta \leq 1.0$ $b_0/t_0 \leq 30$	
Parameters b_{eff}, $b_{e,p}$ *and* k_m	
$b_{eff} = \dfrac{10}{b_0/t_0} \dfrac{f_{y0}t_0}{f_{y1}t_1} b_1$ but $b_{eff} \leq b_i$	For $n > 0$ (compression): $k_n = 1.3 - \dfrac{0.4n}{\beta}$ but $k_n \leq 1.0$
$b_{e,p} = \dfrac{10}{b_0/t_0} b_1$ but $b_{e.p} \leq b_i$	For $n \leq 0$ (: $k_n = 1.0$
*) Fillet welded connections should be designed in accordance with 59.8.	

Table A-9-11: Design resistance moment of welded joints between RHS members and RHS seams

T and X joints	Design resistance
In-plane moments ($\theta = 90°$)	Chord face failure $\beta \leq 0.85$
	$M_{ip,1,Rd} = k_n f_{y0} t_0^2 h_1 \left(\dfrac{1}{2\eta} + \dfrac{2}{\sqrt{1-\beta}} + \dfrac{\eta}{1-\beta} \right) / \gamma_{M5}$
	Chord side wall flattening $0.85 < \beta \leq 1.0$
	$M_{ip,1,Rd} = 0.5 f_{yk} t_0 (h_1 + 5t_0)^2 / \gamma_{M5}$ $f_{yk} = f_{y0}$ for T joints $f_{yk} = 0.8 f_{y0}$ for X joints
	Brace failure $0.85 < \beta \leq 1.0$
	$M_{ip,1,Rd} = f_{y1}\left(W_{pl,1} - (1 - b_{eff}/b_1) b_1 (h_1 - t_1) t_1\right) / \gamma_{M5}$

<table>
<tr><td>Out-of-plane moments ($\theta = 90°$)</td><td>Chord face failure $\beta \leq 0.85$</td></tr>
<tr><td rowspan="7">

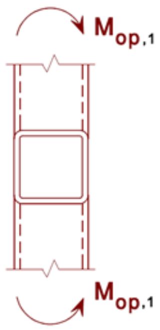

</td><td>$$M_{0p,1,Rd} = k_n f_{y0} t_0^2 \left(\frac{h_1(1+\beta)}{2(1-\beta)} + \sqrt{\frac{2b_0 b_1(1+\beta)}{1-\beta}} \right) / \gamma_{M}$$</td></tr>
<tr><td>Failure of the lateral seam face $0.85 < \beta \leq 1.0$</td></tr>
<tr><td>$$M_{0p,1,Rd} = f_{yk} t_0 (b_0 - t_0)(h_1 + 5t_0) / \gamma_{M5}$$
$f_{yk} = f_{y0}$ for T joints
$f_{yk} = 0.8 f_{y0}$ for X joints</td></tr>
<tr><td>Chord distortional failure (only T joints)*)</td></tr>
<tr><td>$$M_{0p,1,Rd} = 2 f_{y0} t_0 \left(h_1 t_0 + \sqrt{b_0 h_0 t_0 (b_0 + h_0)} \right) / \gamma_{M5}$$</td></tr>
<tr><td>Brace failure $0.85 < \beta \leq 1.0$</td></tr>
<tr><td>$$M_{0p,1,Rd} = f_{y1} \left(W_{pl,1} - 0.5 (1 - b_{eff} / b_1)^2 b_1^2 t_1 \right) / \gamma_{M5}$$</td></tr>
<tr><td colspan="2">Parameters b_{eff} and k_n</td></tr>
<tr><td>$b_{eff} = \frac{10}{b_0 / t_0} \frac{f_{y0} t_0}{f_{y1} t_1} b_1$ but $b_{eff} \leq b_1$</td><td>For $n > 0$ (compression):
$k_n = 1.3 - \frac{0.4n}{\beta}$
but $k_n \leq 1.0$
For $n \leq 0$ (tension): $k_n = 1.0$</td></tr>
<tr><td colspan="2">*) This criterion does not apply where chord distortional failure of the is avoided by other means.</td></tr>
</table>

Table A-9-12: Design criteria for special types of welded joints between RHS members and RHS

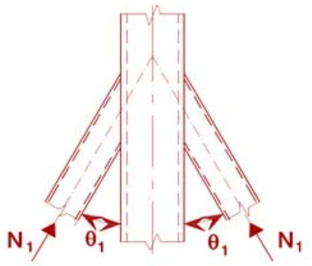
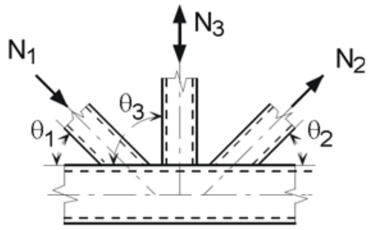
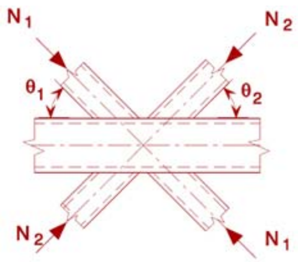
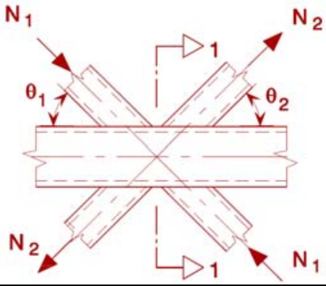
Joint type	Design criteria
The members may be in either tension or compression, and should act in the same direction for both members 	$N_{1,\mathrm{Ed}} \leq N_{1,\mathrm{Rd}}$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for an X joint from Table A-9-8.
Member 1 is always in compression, and member 2 is always intension 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{3,\mathrm{Ed}} \sin\theta_3 \leq N_{1,\mathrm{Rd}} \sin\theta_1$ $N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{1,\mathrm{Rd}} \sin\theta_1$ where $N_{1,\mathrm{Rd}}$ is the value of $N_{1,\mathrm{Rd}}$ for a K joint from Table A-9-9, but with $\dfrac{b_1+b_2+h_1+h_2}{4b_0}$ replaced by: $\dfrac{b_1+b_2+b_3+h_1+h_2+h_3}{6b_0}$
All the diagonals be either compression or 	$N_{1,\mathrm{Ed}} \sin\theta_1 + N_{2,\mathrm{Ed}} \sin\theta_2 \leq N_{x,\mathrm{Rd}} \sin\theta_x$ where $N_{x,\mathrm{Rd}}$ is the value of $N_{x,\mathrm{Rd}}$ for an X joint from Table A-9-8, and $N_{x,\mathrm{Rd}} \sin\theta_x$ is the larger of: $\lvert N_{1,\mathrm{Rd}} \sin\theta_1 \rvert$ and $\lvert N_{2,\mathrm{Rd}} \sin\theta_2 \rvert$
Member 1 is always in compression, and member dl 2 is always in tension 	$N_{i,Ed} \leq N_{i,Rd}$ where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K joint from Table A-9-9. In a gap-type joints seam at section 1-1 the chord satisfies : $\left[\dfrac{N_{0,Ed}}{N_{pl,0,Rd}}\right]^2 + \left[\dfrac{V_{0,Ed}}{V_{pl,0,Rd}}\right]^2 \leq 1.0$

Table A-9-13: Design criteria for, welded joints and joints nt RHS

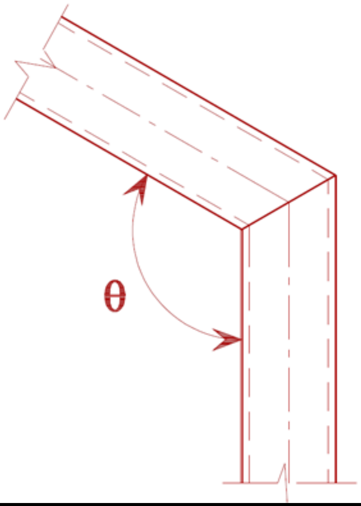
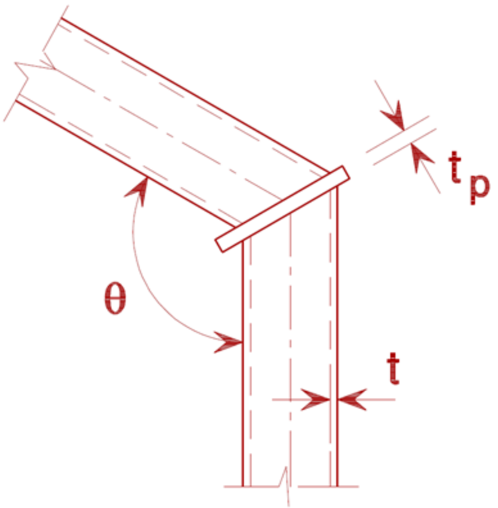
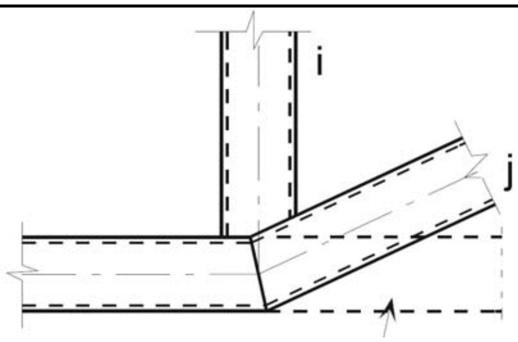
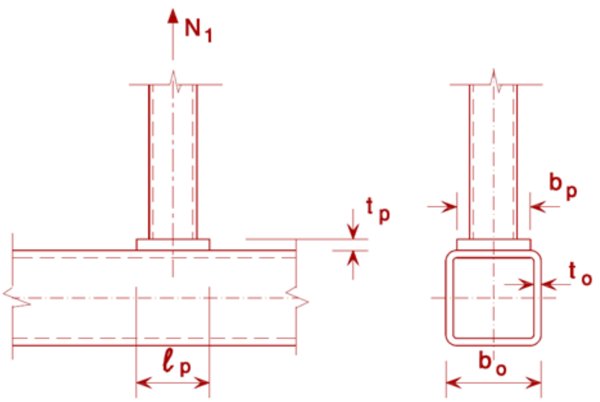
Joint type	Criteria
, welded knee joints	
	The cross-section should be of class 1 for pure bending. $N_{Ed} \leq 0.2 N_{p\ell,Rd}$ and $\frac{N_{Ed}}{N_{pl,Rd}} + \frac{M_{Ed}}{M_{pl,Rd}} \leq k$ If $\theta \leq 90: k = \frac{3\sqrt{b_0/h_0}}{[b_0/h_0]^{0.8}} + \frac{1}{1+2b_0/h_0}$ If $90 < \theta \leq 180$: $k = 1 - \sqrt{2}\cos(\theta/2)(1-k_{90})$ where κ_{90} is the value of κ for $\theta = 90°$.
	$t_p \geq 1.5t$ and $\geq 10\ mm \leq 1.0$ $\frac{N_{Ed}}{N_{pl,Rd}} + \frac{M_{Ed}}{M_{pl,Rd}} \leq 1.0$
Cranked-chord	
 Imaginary extension of chord	$N_{i,Ed} \leq N_{i,Rd}$ where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K or N joint with overlap from th Table A-9-9.

Table A-9-14: Design resistance of reinforced welded T, Y and X joints between RHS or CHS members and RHS chords

<table>
<tr><th>Joint type</th><th>Design resistance</th></tr>
<tr><td colspan="2">Reinforced with flanged plates to avoid chord face failure, failure of, or punching shear.</td></tr>
<tr><td>Tension loading</td><td>$\beta_p \leq 0.85$</td></tr>
<tr><td>
</td><td>

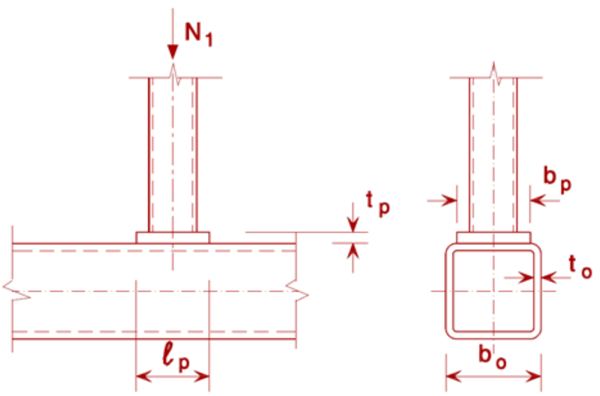
$$\ell_p \geq \frac{h_1}{\sin\theta_1} + \sqrt{b_p(b_p - h_1)}$$

with $b_p \geq b_0 - 2t_0$

and $t_p \geq 2t_1$

$$N_{1,Rd} = \frac{f_{yp} t_p^2}{(1 - b_1/b_p)\sin\theta_1} \cdot \ldots$$

$$\ldots \cdot \left(\frac{2h_1/b_p}{\sin\theta_1} + 4\sqrt{1 - b_1/b_p} \right) / \gamma_{M5}$$

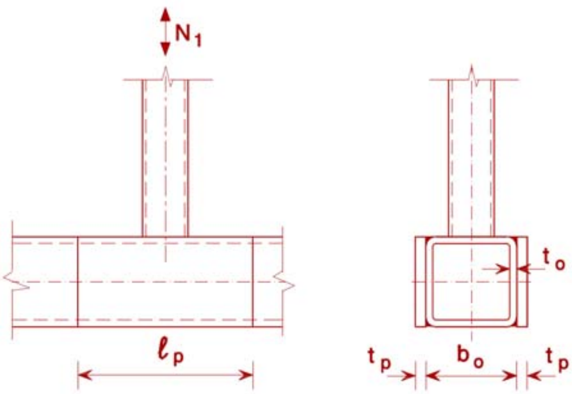
</td></tr>
<tr><td>Compression loading</td><td>$\beta_p \leq 0.85$</td></tr>
<tr><td>
</td><td>

$$\ell_p \geq \frac{h_1}{\sin\theta_1} + \sqrt{b_p(b_p - h_1)}$$

with $b_p \geq b_0 - 2t_0$

and $t_p \geq 2t_1$

Take $N_{1,Rd}$ as the value of $N_{1,Rd}$ for a T, X or Y joint from Table A-9-8, but with k_n = *1.0* and replaced by t_0 t_p for chord face failure, the brace member or punching shear only.

</td></tr>
<tr><td colspan="2">Reinforced with side plates to avoid chord side wall or chord side wall shear buckling failure .</td></tr>
<tr><td>
</td><td>

$\ell_p \geq 1.5 h_1 / \sin\theta_1$

with $t_p \geq 2t_1$

Take $N_{1,Rd}$ as the value of $N_{1,Rd}$ for a T, X or Y joint from Table A-9-8, but replacing t_0 with $(t_0 + t_p)$ only for chord side wall buckling failure chord side wall failure only and shearing .

</td></tr>
</table>

Table A-9-15: Design resistance of reinforced, welded K and N joints between RHS or CHS brace members and RHS chords

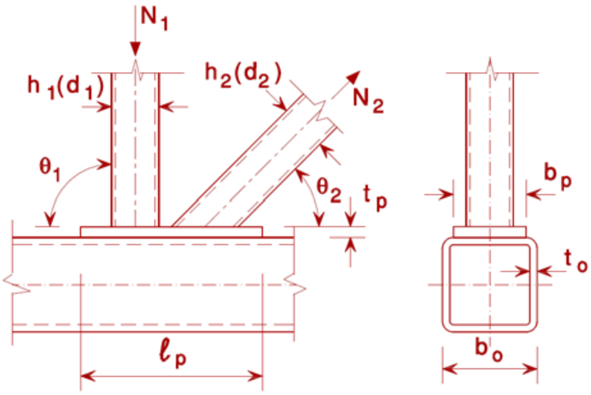
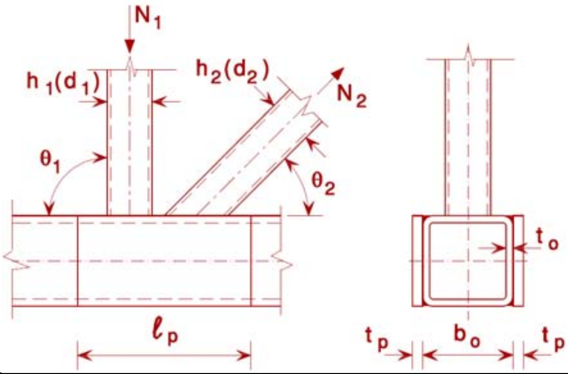
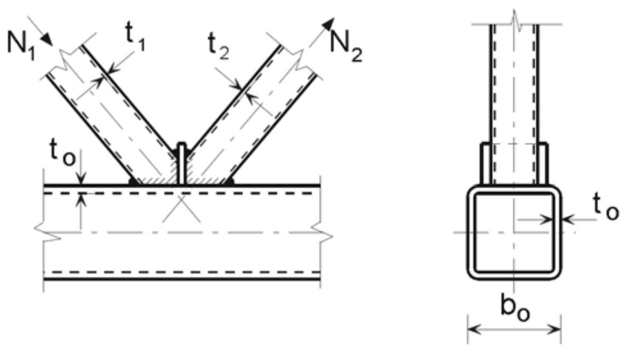
<table>
<tr><td>Joint type</td><td>Design resistance [i = 1 or 2]</td></tr>
<tr><td colspan="2">Reinforced with flanged plates to avoid chord face brace failure, failure of, or punching.</td></tr>
<tr><td>
</td><td>$$\ell_p \geq 1.5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$$
$$b_p \geq b_0 - 2t_0$$
$$t_p \geq 2t_1$$
Take $N_{\mathrm{i,Rd}}$ as the value of $N_{\mathrm{i,Rd}}$ for a K or N joint from Table A-9-9, but replacing t_0 with t_p only f chord face failure, the diagonal or upright,and punching shear only.</td></tr>
<tr><td colspan="2">Reinforcement with a pair of side plates to avoid chord shear failure.</td></tr>
<tr><td>
</td><td>$$\ell_p \geq 1.5\left(\frac{h_1}{\sin\theta_1} + g + \frac{h_2}{\sin\theta_2}\right)$$
Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from Table A-9-9, but replacing t_0 with (t_0+t_p) for chord shear failure only</td></tr>
<tr><td colspan="2">Reinforced by a division plate between the brace members because of insufficient overlap.</td></tr>
<tr><td>
</td><td>$t_p \geq 2t_1$ and $2\,t_2$
Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint with overlap from Table A-9-9, with λ_{ov} < $80\,\%$, and expressing $b_{e,ov}$ as in Table A-10-7, but replacing b_j , t_j and f_{yj} with b_p , t_p and f_{yp}.</td></tr>
</table>

Table A-9-16: Reduction factors for multiplanar joints

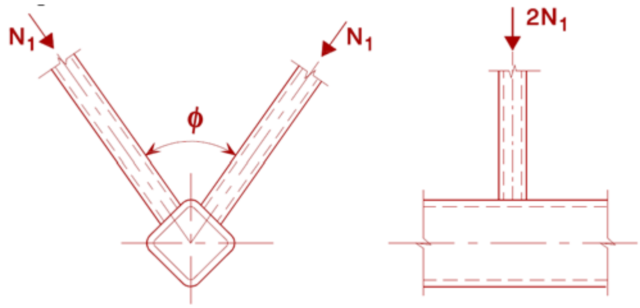
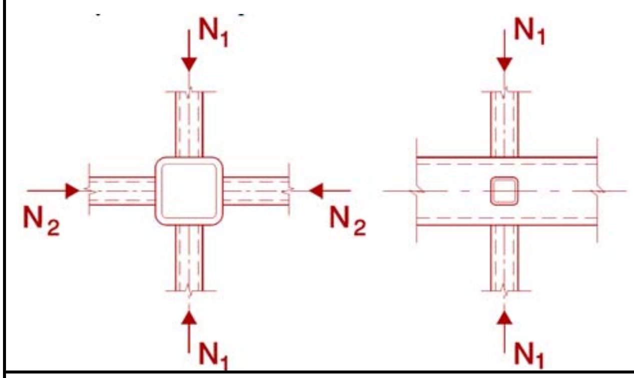
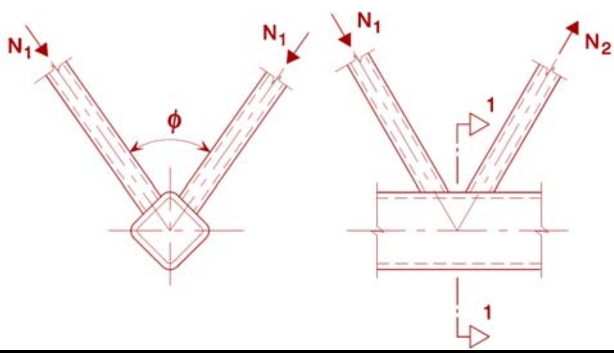
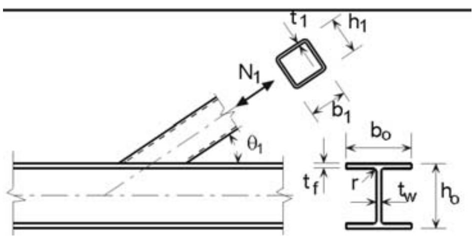
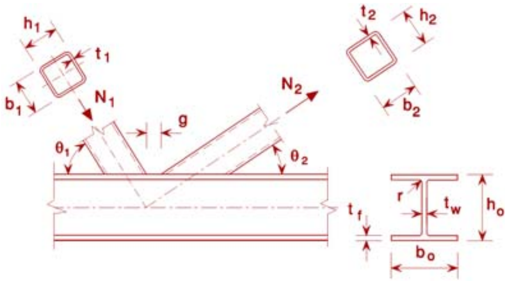
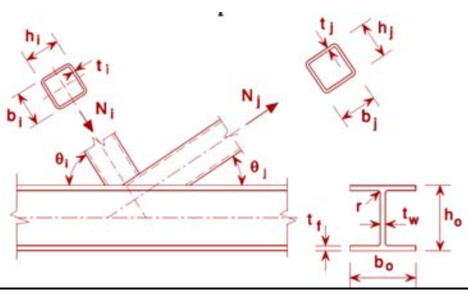
Joint type	Reduction factor μ
TT joint	$60° \leq \varphi \leq 90°$
l Member1 may be in either tension or compression. 	$\mu = 0.9$
XX joint	
Members 1 and 2 can m be either tension or compression. $N_{2,Ed}/N_{1,Ed}$ is negative if one member is in traction and one is in compression. 	$\mu = 0.9(1 + 0.33 N_{2,Ed} / N_{1,E}$ taking account of the sign of $N_{1,Ed}$ and $N_{2,Ed}$ where $\lvert N_{2,Ed} \rvert \leq \lvert N_{1,Ed} \rvert$
KK joint $60° \leq \varphi \leq 90°$	
	$\mu=0.9$ Provided that in a gap-type joint at 1 section 1-1 the chord satisfies : $\left[\frac{N_{0,Ed}}{N_{pl,0,Rd}}\right]^2 + \left[\frac{V_{0,Ed}}{V_{pl,0,Ed}}\right]^2 \leq 1$

Table A-9-17: Design resistance of welded joints between CHS or RHS brace members and I or H section chords

<table>
<tr><td>Joint type</td><td colspan="2">Design Resistance [i = 1 or 2, j = overlapped brace</td></tr>
<tr><td>T, Y and X joints</td><td colspan="2">Chord web Failure</td></tr>
<tr><td rowspan="3">
</td><td colspan="2">$$N_{1,Rd} = \frac{f_{y0} t_w b_w}{\sin \theta_1} / \gamma_{M5}$$</td></tr>
<tr><td colspan="2">Brace failure</td></tr>
<tr><td colspan="2">$$N_{1,Rd} = 2 f_{y1} t_1 p_{eff} / \gamma_{M5}$$</td></tr>
<tr><td>K and Ngap joints [i = 1 or 2]</td><td>Chord stability web</td><td rowspan="4">Brace failure need not be verified if:
$g/t_f \leq 20 - 28\beta$; $\beta \leq 1.0 - 0.03\gamma$ where $\gamma = b_0/2t_f$
and for CHS: $0.75 \leq d_1 / d_2 \leq 1.33$ or for RHS: $0.75 \leq b_1 / b_2 \leq 1.33$</td></tr>
<tr><td rowspan="5">
</td><td>$$N_{1,Rd} = \frac{f_{y0} t_w b_w}{\sin \theta_1} / \lambda_{M5}$$</td></tr>
<tr><td>Brace failure</td></tr>
<tr><td>$$N_{i,Rd} = 2 f_{yi} t_i p_{eff} / \gamma_{M5}$$</td></tr>
<tr><td colspan="2">Chord shear</td></tr>
<tr><td colspan="2">$$N_{i,Rd} = \frac{f_{y0} A_v}{\sqrt{3} \sin \theta_1} / \lambda_{M5}$$
$$N_{i,Rd} = \left[(A_0 - A_v) f_{y0} + A_v f_{y0} \sqrt{1 - (V_{Ed} / V_{p1,Rd})^2} \right] / \gamma_{M5}$$</td></tr>
<tr><td>K and N joints with overlap*) [i = 1 or 2]</td><td colspan="2">Brace failure $25\ \% \leq \lambda_{ov} < 50\ \%$</td></tr>
<tr><td rowspan="5">sMembers i and j may be in either tension or compression.

</td><td colspan="2">$$N_{i,Rd} = f_{yi} t_i (p_{eff} + b_{e,ov} + 2 h_i \lambda_{ov} / 50 - 4 t_i) / \gamma_{M5}$$</td></tr>
<tr><td colspan="2">Brace failure $50\ \% \leq \lambda_{ov} < 80\ \%$</td></tr>
<tr><td colspan="2">$$N_{i,Rd} = f_{yi} t_i (p_{eff} + b_{e,ov} + 2 h_i - 4 t_i) / \gamma_{M5}$$</td></tr>
<tr><td colspan="2">*Brace failure* $\lambda_{ov} \geq 80\ \%$</td></tr>
<tr><td colspan="2">$$N_{i,Rd} = f_{yi} t_i (p_{eff} + b_{e,ov} + 2 h_i - 4 t_i) / \gamma_{M5}$$</td></tr>
</table>

$A_v = A_0 - (2 - \alpha)\, b_0 t_f + (t_w + 2r)\, t_f$ For RHS braces : $\alpha = \sqrt{\dfrac{1}{1 + 4g^2 / (3t_f^2)}}$ For CHS braces : $\alpha = 0$	$p_{eff} = t_w + 2r + 7t_f f_{y0} /$ but for T or Y joints, or for K and N joints with portion: $p_{eff} \leq b_i + h_i - 2t_i$ and for K and N joints overlap : $p_{eff} \leq b_i$	$b_w = \dfrac{h_i}{\sin\theta_i} + 5(t_f + r)$ but $b_w \leq 2t_i + 10(t_f + r)$
	but $b_{e,ov} \leq b_i$	
For CHS braces multiply, the resistances above for brace failure and by $\pi/4$, replacing both b_1 and h_1 by d_1 and b_2, and h_2 by d_2, except for shearing force of the seam. *) It is only necessary to test the brace meber that overlaps i. The efficiency (i.e. the design resistance of the joint divided by the design plastic resistance of the brace member) of the overlapped *by j* should be taken as equal to that of the overlapping brace member. See also Table 64.8.		

Table A-9-18: Design moment resistance of welded joints between rectangular hollow section brace members and I or H section chords

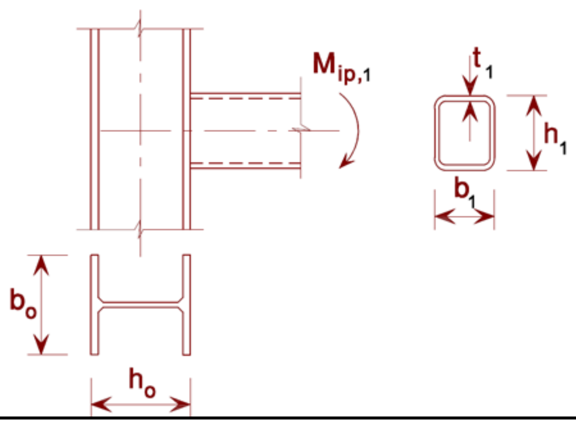
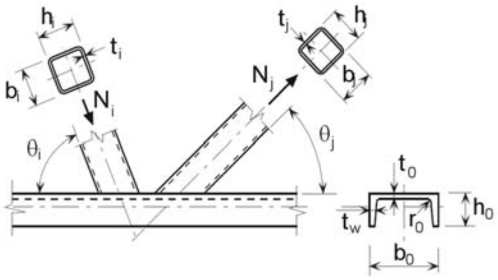
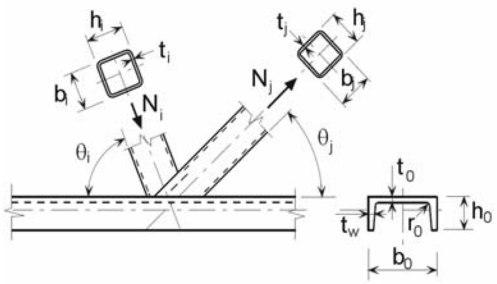
Joint type	Design resistance [i = 1 or 2, j = overlapped brace]
T and Y joints	Failure of chord web yielding
	$M_{ip,1,Rd} = 0.5 f_{y0} t_w b_w (h_1 - t_1) / \gamma_{M5}$ Brace failure of the $M_{ip,1,Rd} = f_{y1} t_1 p_{eff} h_z / \gamma_{M5}$ where h_z is the distance between the centres of gravity of the effective parts of the RHS beam
Parameters p_{eff} and b_w	
$p_{eff} = t_w + 2r + 7t_f f_{y0} / f_{y1}$ but $p_{eff} \leq b_i + h_i - 2t_i$	$b_w = \dfrac{h_1}{\sin\theta_1} + 5(t_f + r)$, but $b_w \leq 2t_1 + 10(t_f + r)$

Table A-9-19: Design resistance of welded joints between CHS or RHS brace members and U section chords

<table>
<tr><td>Joint type</td><td>Design resistance [i = 1 or 2, j = overlapped diagonal or upright]</td></tr>
<tr><td>K and N joints with separation</td><td>Brace failure</td></tr>
<tr><td rowspan="3">
</td><td>$$N_{i,Rd} = f_{yi} t_i (b_i + b_{eff} + 2h_i - 4t_i) / \gamma_{M5}$$</td></tr>
<tr><td>Failure of the seam</td></tr>
<tr><td>$$N_{i,Rd} = \frac{f_{y0} A_v}{\sqrt{3} \sin \theta_1} / \gamma_{M5}$$
$$N_{0,Rd} = A_0 - A_v f_{y0} + A_v f_{y0} \sqrt{1 - \left(V_{Ed} / V_{pl,Rd}\right)^2} / \gamma_{M5}$$</td></tr>
<tr><td>K and N joints with overlap *)</td><td>Brace failure $25\ \% \le \lambda_{ov} < 50\ \%$</td></tr>
<tr><td rowspan="6">
</td><td>$$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + \left(2h_i \frac{\lambda_{ov}}{50} - 4t_i \right) \right) / \gamma_{M5} \mathrm{N}$$</td></tr>
<tr><td>Brace failure $50\ \% \le \lambda_{ov} < 80\ \%$</td></tr>
<tr><td>$$N_{i,Rd} = f_{yi} t_i \left(b_{eff} + b_{e,ov} + 2h_i - 4t_i \right) / \gamma_{M5}$$</td></tr>
<tr><td>Brace failure $\lambda_{ov} \ge 80\ \%$</td></tr>
<tr><td>$$N_{i,Rd} = f_{yi} t_i \left(b_i + b_{e,ov} + 2h_i - 4t_i \right) / \gamma_{M5}$$</td></tr>
<tr><td></td></tr>
<tr><td>$A_v = A_0 - (1 - \alpha)\, b_0^* t_0$
$b_0^* = b_0 - 2\,(t_w + r_0)$
For RHS: $\alpha = \sqrt{\frac{1}{1 + 4g^2 / 3t_f^2}}$
For CHS: $\alpha = 0$
$V_{pl,Rd} = \frac{f_{y0} A_v}{\sqrt{3}} / \gamma_{M5}$
$V_{Ed} = (N_{i,Ed} \sin \theta_i)_{max}$</td><td>$b_{eff} = \frac{10}{b_0^* / t_0} \frac{f_{y0} t_0}{f_{yi} t_i} b_i$, but $b_{eff} \le b_i$
$b_{e,ov} = \frac{10}{b_j / t_j} \frac{f_{yj} t_j}{f_{yi} t_i} b_i$, but $b_{e,ov} \le b_i$</td></tr>
<tr><td colspan="2">For CHS braces except the chord failure, the resistances above are multiplied by $\pi/4$, and replace both b_1 and h_1 by d_1 as well b_2, and h_2 by d_2, e</td></tr>
<tr><td colspan="2">*) only the overlapping brace member i needs to be checked i. The efficiency (i.e. the design resistance of the joint divided by n the stic design plastic resistance of the member) of the overlapped member j should betaken as equal to that of the overlapping brace member</td></tr>
</table>

Annex 10: Guarantee levels and requirements for official recognition of quality marks

A10.1 Introduction

This Code specifies Project Management's ability to apply special considerations to certain products that voluntarily have additional levels of guarantee over and above the minimum required by the regulations, in accordance with Section 84.

In general, these additional levels of guarantee are shown through the possession of a quality mark that has been officially recognised by a competent government body in the field of construction in a Member State of the European Union, any of the signatory states to the Agreement on the European Economic Area, or any state that has an agreement with the European Union on the establishing of a customs union, in which case the level of equivalence shall be found by applying the procedures established in Directive 89/106/EEC.

A10.2 Guarantee levels for products

In case of products that need to have CE marking in accordance with Directive 89/106/EEC, the level of guarantee required by the regulations is that associated with the CE marking, as specified in the corresponding harmonised European standards, and which allows free trade within the European Economic Area. In the case of products for which there is no CE marking in force, the level of guarantee required by the regulations is that set out in the body of this Code.

Additionally, the Manufacturer of any product may voluntary opt for a quality mark that ensures a guarantee level that exceeds the minimum requirement laid down by this Code.. In the case of products with the CE marking, such quality marks should provide an added value in relation to properties not covered d by this marking.

Where they are voluntary, quality marks may show different criteria for concession in the corresponding special procedures. This Annex sets out the conditions that allow a distinction to be made as to when there is an additional level of guarantee in relation to the minimum required by the regulations, and which may therefore be recognised officially by the competent National authority.

A10.3 Technical bases for official recognition of quality marks

The national authority body that officially recognises the quality mark must verify that it fulfils the requirements for official recognition contained in this Annex and that such fulfilment is maintained. In order to achieve this objective,

The Administration may, in observance of the necessary confidentiality, intervene in any activities that it deems relevant for recognition of marking.

The official regulation in which the competent government body awards recognition must state explicitly that recognition is done for the purposes of this Code and in accordance with the technical bases contained in this Annex.

The national authority that officially recognises a quality mark for a product or process may, in order to check whether the requirements have been fulfilled, require that designated representatives to participate in the committees defined in the certification body for taking decisions on certification materials. The national authority shall have access to all documentation relating to the mark, and shall make the appropriate confidentiality commitments.

A10.4 General requirements for quality mark

For its official recognition, a mark shall:

- be of a voluntary nature and awarded by a certification body that fulfils the requirements of this Annex;

- be in agreement with this Code and to include an explicit declaration of such compliance in its rules;

- be awarded based on a procedure described in a governing regulation for the quality mark that defines its specific guarantees, the procedure for awarding it, the operating system, the technical requirements and the rules concerning decision-making relating to it. This regulation should be available to the public, defined in clear, precise terms, and should provide unambiguous information, both for the certification customer and for other interested parties. Such regulation shall also consider specific procedures, both in the case of external installations to the construction site and installations that belong to the site, as well as processes developed in site.

- guarantee independence and impartiality in the concession, not allowing, among other measures, taking part in the decisions regarding any expedient to persons developing consultancy activities related to it;

- include, in the regulations governing the granting of the quality mark, the corresponding treatment for certified products, presenting the results of non-compliant production tests, so as to guarantee that appropriate corrective action is taken immediately and, if applicable, clients are informed. The rules shall also lay down the maximum period of time that may elapse between the non-compliance being detected and l the corrective action that being taken;

- lay dawn the minimum requirements that must be fulfilled by laboratories working on certification;

- establish for the awarding that shall be a continuous production control during a period of at least six months for products developed at facilities off site. In the case of facilities on site, the governing regulations shall consider criteria for ensuring the same level of production information and guarantees for the user;

- present additional guarantees for properties that are separate from the regulatory requirements but which may contribute to compliance with the requirements of this Code, in case of products or processes not included in this Annex but included in the scope of this Code.

A10.5 Specific requirements of quality marks

This Code defines, besides the general requirements in section 4 of this Annex, defines some specific requirements that should fulfil by quality marks in order to be recognised by an Administration.

A10.5.1 Members manufactured at the workshop

The quality mark shall t:

- guarantee that the reception of steel products and joint members used, where applicable, as well as the stocks system, allow for perfect traceability by means of continuous documented control and of consumption of such products;

- demand a computerised control system for the traceability of members regarding to the products used;

- demand that where there are discontinuities in production of more than 1 month, the manufacturer shall communicate the same to the certification body. The manufacturer shall otherwise be subject to penalties, which must be set out in the regulation of the quality mark. The requirements for the production and intensity of controls after the discontinuity must be set out in the regulations, depending on the causes of such interruption;

- require workshops to have labelling systems through computerised codes to guarantee traceability of the members and which allow for subsequent traceability management on site;

- define and apply, where relevant, penalties that guarantee the minimum impact to the user in the event of the production of non-conformity products. To this end, than 3 months since the detection of non-conformity relating to the product requirements until the suspension of use of the mark for such certified product, if the non-conformity c has not been settled r by then.

A10.5.2 Steel products

The quality mark for steel products shall:

- guarantee added value with regard to characteristic not covered by the CE marking;

- guarantee added value focused on the transformation processes at assembly workshops;

- guarantee product properties that are consistent with the special considerations discussed for such cases in this Code;

- require that manufacturers to have labelling systems by means of computerised codes to guarantee traceability of the steel up to casting level and which allow such traceability to be managed by the customer.

A10.6 General responsibilities of certification bodies

This shall be a body accredited in accordance with Regulation (CE) 765/2008 of the European Parliament and the Council, of 9 July 2008, according with UNE-EN 45011 for the certification of products.

The certification body shall make available to the national authority body that carries out recognition all the necessary information for the correct development of activities for which it is competent in relation to the recognition of marks.

The certification body should also:

- notify the national authority body that carries out official recognition of any changes to the initial conditions under which recognition was granted;

- have a specific body for each product that analyses the application of the regulation and adopts or, where relevant, proposes the adoption of decisions relating to the award of the mark. It must include equal numbers of representatives of manufacturers, users and certification partners (laboratories, auditors, etc.);

- check that the laboratory used to carry out production controls has adequate material and human resources;

- check conformity of production control test results with a periodicity adequate for the manufacture of the product and not, under any circumstances, less than once every six months. The regulations shall therefore set out acceptance criteria, both for statistics and for individual ones. In order to analyse these test results, the regulations shall also set out the criteria for correcting them, depending on the results obtained by the verification laboratory in the contrast tests. Statistical conformity of both corrected and uncorrected self-control results must be checked;

- check that, in the event of a non-conformity production control, manufacturers have taken corrective measures within a period of not more than one week, and have notified their customers in writing with the results of the self- control. Any non-conformity must be resolved within a maximum period of three months. Depending on whether corrective measures are adopted, an additional period of three months may be granted,

  at the end of which period the mark shall be withdrawn if the non-conformity still exists. In this case, and in order to adopt measures quickly, the manufacturer's allegations and the proposal to withdraw the mark may, where relevant, be carried out using computerised processes (Internet, etc.);

- use verification laboratories to carry out periodic contrast tests of the properties of the products protected by the mark. The sampling to carry out these tests must be done guaranteeing that they are representative and distributed properly to verification laboratories and to the manufacturers' own laboratories, where relevant. The certification body shall, depending on the results obtained, make corrections to the data obtained from the production control, where relevant;

- organise round-robin testing programmes at least once a year, to allow tracking the evolution of the verification laboratories;

- set up a market surveillance, such that all products protected by the mark are subject to periodic analysis by taking samples for testing and checking that in all cases the documentation guarantees both traceability and consistency of the supplied product with the characteristics of the product that appear in the supply record.

A10.7 Requirements for verification laboratories

These should be the certification body's own laboratories, or subcontracted laboratories, accredited in accordance with Regulation (CE) 765/2008 of the European Parliament and the Council, of 9 July 2008, in accordance with UNE-EN ISO/IEC 17025 or belonging to a government body with competence in the field of construction, as discussed in subsection 78.2.2.1.

A10.8 Requirements relating to the production premises

The production facility shall:

- have implemented a quality management system that is audited by an authorized certification body implemented accredited in accordance with Regulation (CE) 765/2008 of the European Parliament and the Council, of 9 July 2008, in accordance with UNE-EN ISO/IEC 17021. This system must comply with the relevant parts of standard UNE-EN ISO 9001;

- have a laboratory for continuous control of production and of the product to supply, whether this its own laboratory or a contracted one;

- have defined and implemented continuous production control at the factory, the data from which must be available for a period of at least six months before the concession is granted. This period may be two months in some special cases where the same product is manufactured regularly, such as at the work site facilities, for example. For such cases, the regulations of the mark shall include specific criteria that ensure the same level of guarantee for the user as in general case, so that the mark may be granted within a maximum period of two months from the submission of the self-control data referred to above;

- have a signed insurance policy of a sufficient sum that offers protection for its civil liability for possible faulty products manufactured, in accordance with the regulations for the quality mark;

- have an information system for production control results, which shall be accessible for the user by means of computerised procedures (Internet, etc.).

Annex 11: Structure's contribution to sustainability index

A11.1 General

The design, execution and maintenance of steel structures are activities that fall under the general context of construction and which may contribute to fulfilment of the conditions that allow for adequate sustainable development.

Sustainability is a global concept that is not specific to steel structures, and it requires a series of environmental financial and social criteria be satisfied. The contribution of steel structures to sustainability therefore depends on compliance with the criteria such as reasonable use of the energy employed (both for the manufacture of construction products and for progression of execution), the use of renewable resources, the use of recycled products and minimising the impact on the environment as a result of execution and the creation of healthy work areas. The design, execution and maintenance of steel structures may also take account of other aspects, such as amortising the initial impact over the working life of the structure, optimising maintenance costs, incorporating innovative technology resulting from corporate RDI strategies, continuously training the staff participating in the stages of the structure, or other aspects of a financial or social nature.

Without prejudice to compliance with the environmental protection legislation in force, the Owner may stipulate that the structure's execution consider a series of environmental considerations, so as to minimise the potential impact of such activity. In such case, this requirement should be included in an environmental assessment Annex for the structure, to form part of the design. In case the design does not consider requirements of this type for the execution phase, the Owner may demand an obligation of compliance by inserting the relevant clauses in the contract with the Builder.

When the Designer has set out requirements relating to the contribution of the structure to sustainability, and the Owner so decides, this Annex defines the structure's sustainability contribution index (ICES-EA) [Index of Contribution to Sustainability by a Steel Structure], obtained using the environmental sensitivity index therein (ISMA-EA) [index of environmental sensitivity of a steel structure], setting out procedures for estimating them where so decided upon by the Owner.

The criteria mentioned in this Annex apply exclusively to activities relating to steel structures. In the case of members that are frequently included in assemblies on very extensive sites (buildings, industrial sites, bridges, etc.), the Designer and Project Management must ensure,

Where relevant, that these criteria are coordinated in relation to those adopted for the rest of the structural work.

A11.2 General criteria for steel structures

The assessment of sustainability indicators or, where relevant, environmental indicators, discussed in this Code may be intended to:

- compare different structural steel solutions; or

- establish quantitative parameters for evaluating the quality of the structure in relation to such aspects.

In general, a structure has added value in terms of sustainability when it meets the requirements defined in Section 5 of this Code, with:

- optimisation of material consumption, using smaller quantities of steel;

- extension of the working life of the structure, over which it produces greater amortisation of any impact produced during the execution stage ;

- use of steel:

  - originating from the recycling of ferric by-products);

  - obtained using processes that produce lower emissions of CO_2 into the atmosphere;

  - that demonstrates the fact that waste is put to good use, such as blast-furnace , for example;

  - originating from processes that guarantee the use of primary iron materials that do not have any radiological contamination;

- establishment of voluntary environmental certification systems for manufacturing processes relating to all products used in the structure and, in particular, those for the execution of the structure, including workshop manufacture, assembly on site and transportation to the site, where relevant;

- use of products with officially recognised quality marks ensuring that the basic requirements of the structure are adequately met with the least possible degree of uncertainty − conformity with additional preventive criteria to those set out in the regulation in force and which may apply to health and safety on the site;

- application of innovative criteria that increase productivity, competitiveness and efficiency of construction, as well as user accessibility;

- minimising potential impact on the environment resulting from execution of the structure (noise, particles, etc.); and

- the smallest possible use of natural resources in general.

A11.3 General method for considering sustainability criteria

The consideration of sustainability criteria for a steel structure shall be decided upon by the Owner, who must also:

- communicate the same to the Designer so that he can incorporate the corresponding measures when drawing up the design;

- consider this when commissioning execution;

- check that the Builder complies with the criteria during execution; and

- ensure that appropriate maintenance criteria are conveyed to users, where relevant.

The Owner must, if appropriate, communicate the Designer of the sensitivity criterion with which the building must comply in accordance with section A11.5 of this Annex. A steel structure shall be considered as fulfilling the criterion defined by the Owner where the following conditions are fulfilled, where relevant:

$$\text{ICES-EA}_{\text{property}} \leq \text{ICES-EA}_{\text{design}} \leq \text{ICES-EA}_{\text{execution}}$$

where:

- Owner — indicates that the ICES-EA index is defined by the Owner in the commissioning;
- Design — indicates that this is the index established by the Designer;
- Execution — indicates that the index has been obtained as a result of the control on the actual conditions under which the structure has been executed, in accordance with Section 91 of this Code.

A11.4 Environmental sensitivity index of the steel structure (ISMA-EA)

A11.4.1 Definition of the environmental sensitivity index (ESI)

The “environmental sensitivity index” of a structure is defined as the result of applying the following expression:

$$ISMA - EA = \sum_{i=1}^{i=6} \alpha_i \beta_i \gamma_i V_i$$

where:

α_i, β_i *and* γ_i weighting factors for each requirement, criterion or indicator in accordance with Table A11.4.1.a;

V_i factors for the value obtained for each criterion, in accordance with the following expression, depending on the representative parameter in each case:

$$V_i = k_i \left[1 - e^{m_i \left(\frac{P_i}{n_i} \right)^{A_i}} \right]$$

where:

K_i, m_i, n_i and A_i parameters whose values depend on each indicator, in accordance with Table A11.4.1.b;

P_i value representing each indicator, in accordance with section A11.4.3 of this Annex.

Table A11.4.1.a. Weighting factors

Environmental requirement	Weighting factor		
	α	β	γ
Environmental properties of steel products	0.70	0.40	1
Optimisation of execution		0.40	0.5
Level of execution control			0.5
Environmental optimisation of steel		0.20	1
Specific measures for impact control	0.30	0.33	1
Specific measures for waste management		0.67	1

Table A11.4.1.b

Environmental requirement	*Ki*	*mi*	*ni*	*Ai*
Environmental properties of steel products	1.02	-0.50	50	3.00
Optimisation of execution	1.06	-0.45	35	2.50
Level of execution control	1.05	-1.80	40	1.20
Environmental optimisation of steel	10.5	-0.001	1	1.00
Specific measures for impact control	10.5	-0.001	1	1.00
Specific measures for waste management	1.21	-0.40	40	1.60

A11.4.2 Environmental classification of facilities

For the purposes of this Code, a facility shall be understood as having an environmental mark when it is in possession of a quality mark in accordance with UNE-EN ISO 14001 or an EMAS.

Even without an environmental mark, the facility shall be considered to have an environmental commitment for the purposes of this Code where it fulfils the following conditions:

a) In the case of a workshop manufacturing facility:

- it has an officially recognised quality mark, in accordance with Section 84 of this Code;
- it uses steel products that have an officially recognised quality mark;
- it checks and records waste management or recycling processes (e.g. use of containers, waste management plans, etc.);

b) In the case of the company undertaking on-site execution :

- it uses steel products that have an officially recognised quality mark;

- it checks and records waste management or recycling processes (e.g. use of containers, waste management plans, etc.);

- it has demarcated areas for the warehousing of products;

- it adopts measures to reduce noise emissions caused by the processes that take place in structure's execution;

c) In the case of the construction company, in relation to assembly on site:

- it collects waste in separate containers for their recycling and management;

- it has demarcated areas for the warehousing of products;

- it adopts measures to reduce noise emissions caused by the processes that take place in structure's execution.

A11.4.3 Environmental criteria and representative functions

A11.4.3.1 Environmental criterion of characterisation of steel products

This criterion assesses the environmental sensitivity of the manufacture of steel products. It is intended to reduce CO_2 emissions from the manufacture of steel, and to reduce the quantity of waste produced by the manufacture of steel products.

$$P_1 = \frac{1}{100}\frac{A}{100}\sum_{i=1}^{i=2} p_{1i} \cdot \lambda_{1i}$$

where:

λ_{1i} values obtained from Table A11.4.3.1;

A percentage of steel products that have an officially recognised quality mark ;

p_{1i} percentage of on-site use of each type of steel identified in Table A11.4.3.1.

Table A11.4.3.1

Environmental condition	In accordance with / by means of	Value factor (λ_{1i})
No certification	Standard UNE-EN ISO 14001 and the EMAS system do not apply, or the product is not certified by means of a voluntary quality mark with an officially recognised mark , or the product certificate does not attest whether such steel is subject to the requirements of the Kyoto Protocol.	0
With production subject to environmental certification	Standard UNE-EN ISO 14001.	10
	Standard UNE-EN ISO 14001 and EMAS record, or EMAS record without standard UNE-EN ISO 14001.	40
	By virtue of having an officially recognised quality mark, the manufacture of the steel is accredited as being subject to the requirements of the Kyoto Protocol.	60

A11.4.3.2 Environmental criterion of optimisation of execution

This criterion assesses the environmental sensitivity with which the workshop assembly manufacturing processes for structures take place, as well as the procedures for. Its objectives are as follows:

- to reduce the f waste produced by manufacturing;
- to encourage optimisation of members and recycling of any waste the production of which is inevitable; and
- to reduce impact during assembly on site.

The representative function of this criterion is defined by:

$$P_2 = \frac{1}{100}\sum_{i=1}^{i=2} p_{2i} \cdot \lambda_{2i}$$

where p_{2i} is the percentage that represents each of the possible origins of the members used in the structure , and λ_{2i} is the sum of the values that may apply according to the environmental conditions at the facilities, for the corresponding column in Table A11.4.3.2.

Table A.11.4.3.2

Facility	Environmental condition	Factors (λ_{2i})	
		Case 1: Workshop manufacturing facility	Case 2: On-site execution facility
		λ_{21}	λ_{22}
Workshop manufacturing facility	With environmental mark	80	-
	With environmental commitment	60	-
	Other cases	30	-
On-site execution facility	With environmental mark	-	70
	With environmental commitment	-	30
	Other cases	-	0
Construction company	With environmental mark	20	30
	With environmental commitment	10	15
	Other cases	0	0

The values in the table above correspond to a maximum transportation distance of 300 km for members that are manufactured in the workshop. Where this transportation distance is greater, the factor value λ_{21} corresponding to the workshop manufacturing facility shall be reduced by 5, and that corresponding to the construction company shall be increased by 5, with the exception of the “Other cases” row which shall remain as 0.

A11.4.3.3 Environmental criterion of execution control systems

This criterion assesses the environmental contribution associated with the reduction in resources consumed by preparation of the structure, as a result of an intensive execution control level and the use of products that have an officially recognised quality mark.

The representative function of this criterion is defined by:

$$P_3 = \frac{1}{100}\sum_{i=1}^{i=2} p_{3i} \cdot \lambda_{3i}$$

where p_{3i} is the on-site usage percentage for each of the cases defined in Table A11.4.3.3 and λ_{3i} is the factor reflected in the same for each case.

Table A11.4.3.3

Sub-criterion	Value factor (λ_{3i})
No reduction in γ_M applies, in accordance with subsection 15.3	λ_{31}=0
A reduction in γ_M applies, in accordance with subsection 15.3	λ_{32}=100

A11.4.3.4 Environmental criterion of steel optimisation

This criterion assesses the environmental contribution associated with the recycling of ferric by-products as well as whether good use is made of the by-products produced during the process.

The representative function of this criterion is defined by:

$$P_4 = \frac{1}{100}\frac{A}{100}\sum_{i=1}^{i=2} p_{4i} \cdot \lambda_{4i}$$

where:

λ_{4i} values obtained from Table A11.4.3.4 ;

A percentage of steel that is in possession of an officially recognised quality mark;

p_{4i} percentage of on-site use of each type of steel identified in Table A11.4.3.4.

Table A11.4.3.4

Optimisation of resources in steel manufacture	Factors (λ_{4i})
By virtue of its being in possession of an officially recognised quality mark, the manufacture of the steel is accredited as originating from the recycling of waste for at least 80 %.	λ_{41}=45
By virtue of having an officially recognised quality mark, the steel is recognised as making good use of more than 50 % of its dross.	λ_{42}=25
By virtue of its being in possession of an officially recognised quality mark, the steel is recognised as having been subjected to verifiable, documented radiological emissions tests, both for the primary iron materials used in the steel work and for the steel products.	λ_{43}=30

A11.4.3.5 Environmental criterion of impact control

This criterion assesses the environmental contribution associated with the execution of a structure that minimises the impact on the environment and, in particular, the emission of particles and gases into the air.

The representative function of this criterion is defined by:

$$P_5 = \sum_{i=1}^{i=2} p_{5i} \cdot \lambda_{5i}$$

where p_{5i} and λ_{5i} are the parameters obtained from Table A11.4.3.5.

Table A11.4.3.5

Sub-criterion	p_{5i}	λ_{5i}
No painting done on site	1	50
No welding done on site	1	50

A11.4.3.6 Environmental criterion of waste management

This criterion assesses the environmental contribution associated with the execution of a structure that provides adequate management of waste generated during that process. In particular, it takes account of the existence of a management plan for excavation materials, a management plan for construction waste and demolition.

The representative function of this criterion is defined by:

$$P_6 = \lambda_6$$

where λ_{6i} are the values obtained from Table A11.4.3.6.

Table A11.4.3.6

<table>
<tr><td>Sub-criterion</td><td colspan="2">Cases</td><td></td></tr>
<tr><td rowspan="6">Management of construction and demolition waste (RCD).</td><td>No controlled action</td><td></td><td>0</td></tr>
<tr><td rowspan="5">Recycle a percentage, as given in the next column, and the rest to dumps .</td><td>20 %</td><td>20</td></tr>
<tr><td>40 %</td><td>40</td></tr>
<tr><td>60 %</td><td>60</td></tr>
<tr><td>80 %</td><td>80</td></tr>
<tr><td>100 %</td><td>100</td></tr>
</table>

A11.5 Sustainability contribution index of the structure

The “sustainability contribution index of the structure” (ICES-EA) is defined as the result of applying the following expression:

ICES-EA =a+b·ISMA-EA

and the following must also be fulfilled:

ICES-EA ≤ 1

ICES-EA ≤ 2.ISMA-EA

ICES-EA is the minimum of:

1) a+b·ISMA-EA

2) 1

3) ISMA

where:

a social contribution factor, obtained as the sum of the factors given in Table A11.5, in accordance with the sub-criteria that may apply:

$$a = \sum_{i=1}^{i=5} a_i$$

Table A.11.5

Sub-criterion	In design	In execution
The Constructor applies innovative methods resulting from RDI projects carried out in the last 3 years.	a_1 =0	a_1 =0.02
At least 30 % of staff working on execution of the structure have taken specific training courses in technical, quality or environmental aspects.	a_2= 0	a_2= 0.02
Voluntary health and safety measures are adopted in addition to those required by the regulations for execution of the structure.	a_3= 0	a_3= 0.04
A public website, specific to the works, is prepared in order to inform the public, and which includes its characteristics and execution deadlines, as well as its financial and social implications.	a_4 = 0.01	a_4 = 0.02
The structure is included in a structural work declared to be in the public interest by the competent government body.	a_5 = 0.04	a_5 = 0.04

b contribution factor by extension of the working life , obtained in accordance with the following expression:

$$b = \frac{t_g}{t_{g,\min}} \leq 1.25$$

b is the minimum of:

1) $\frac{t_g}{t_{g,\min}}$

2) 1.25

where:

t_g working life actually considered in the design for the structure, within the ranges considered in Section 5 of the Code; and

$t_{g,\min}$ value of the working life set out in subsection 5.1 of the Code for the corresponding type of structure.

On the basis of the ICES-EA, the contribution of the structure to sustainability may be classified in accordance with the following levels:

Level A: 0.81 ≤ ICES-EA ≤ 1.00

Level B: 0.61 ≤ ICES-EA ≤ 0.80

Level C: 0.41 ≤ ICES-EA ≤ 0.60

Level D: 0.21 ≤ ICES-EA ≤ 0.40

Level E: 0.00 ≤ ICES-EA ≤ 0.20

where A is the maximum end of the scale (maximum contribution to sustainability) and E is the minimum end of the scale (minimum contribution to sustainability).

A11.6 Checking the sustainability contribution criteria

A11.6.1 Evaluation of the sustainability contribution index of the structure in the design

In the event that the Owner decides to apply sustainability criteria to the structure, the Designer must define a strategy for complying with them in the design, by assessing the design value of the sustainability contribution index of the structure (ICES-EA$_{design}$) and identifying the criteria, or sub-criteria where relevant, that must be fulfilled in order to achieve the set value.

In order to evaluate the ICES-EA$_{design}$ index, $a_1 = a_2 = a_3 = 0$ shall be used.

Furthermore, the Designer shall also reflect the necessary measures that are to be taken into account during execution of the structure in the corresponding documents and, in particular, in the technical report, the Special Technical Specifications and the budget.

A11.6.2 Evaluation of the actual sustainability contribution index of the structure on execution

Where the Owner has decided to apply sustainability criteria to the structure, Project Management must check, either directly or through a quality control body, that the actual value of the sustainability contribution index of the structure as a result of the actual conditions of execution (ICES-EA$_{execution}$) is not less than the value for that index as defined in the design.

Accreditation documents for the final assessment of ICES-EA$_{execution}$ shall form part of the final work documentation.