TECHNICAL REPORT

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Aluminium structures — Material and design — Ultimate limit state under static loading

Structures en aluminium — Matériaux et conception — État limite ultime sous charge statique

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Contents

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Annexes

Foreword

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

The main task of technical committees is to prepare International Standards, but in exceptional circumstances a technical committee may propose the publication of a Technical Report of one of the following types:

— type 1, when the required support cannot be obtained for the publication of an International Standard, despite repeated efforts;

— type 2, when the subject is still under technical development or where for any other reason there is the future but not immediate possibility of an agreement on an International Standard;

— type 3, when a technical committee has collected data of a different kind from that which is normally published as an International Standard ("state of the art", for example).

Technical Reports of types 1 and 2 are subject to review within three years of publication, to decide whether they can be transformed into International Standards. Technical Reports of type 3 do not necessarily have to be reviewed until the data they provide are considered to be no longer valid or useful.

ISO/TR 11069, which is a Technical Report of type 1, was prepared by Technical Committee ISO/TC 167, *Steel and aluminium structures*, Subcommittee SC 3, *Aluminium structures*.

This Technical Report was proposed as a Draft International Standard but failed to obtain the required committee support. Reasons for the failure were primarily concerned with the attempt to promote the standard prepared by the ECCS. There was no serious technical disagreement; thus the grounds for the lack of support could not be resolved by addressing the technical content.

Aluminium finds wide application in load-carrying assemblies, such as building structures, vehicles and ships, to which the rules for strength design have general validity. This Technical Report deals with the resistance of aluminium structural elements, without regard to any specific product. Because of this broad target, such standards as those developed for steel in particular markets cannot provide a model. No single European standard is suitable for international acceptance; therefore the procedures presented are compromises to satisfy the demands of both Europe and North America.

National standards include the required safety levels for the particular field treated. Because there is no specific field considered in the Report, there can be no values given for resistance factors or other safety margins. However, if the load spectrum and desired reliability are known, safe design procedures are readily obtained using the resistances provided.

Being intended for international use, no purpose is served by further delaying the issuing of the Report by waiting for the final ECCS recommendations, or the conflicting British Standard BS 8118, or the very distant CEN code for aluminium structures.

Design procedures have been based on the current techniques used for steel structures, adjusted to suit aluminium, and the presentation will be familiar to those using modern steel codes of practice, but may differ in some respects from the methods of earlier aluminium standards.

The Commentary to the Report gives only a limited review of the sources of the treatments proposed. Over the 16 years that the Committee has been meeting, a great deal of material has been produced which records the many aspects of each of the subjects that have been examined, and reveals the areas where compromise has been needed.

Annexes A and B of this Technical Report are for information only.

Introduction

Limit states design requires a knowledge of the ultimate load capacities of components and of the distortion of structural assemblies under the action of specified service loads. These two aspects of structural behaviour are in most cases unrelated and require independent treatment.

This Technical Report gives the ultimate load capacity of aluminium members and connections used in stressed applications, under the action of static loads. The types of component treated include bars, plates and panels; the types of connection are riveted, bolted and welded.

Under the action of static loads, some local buckling and local yielding, up to fully plastic behaviour, are acceptable prior to the attainment of the ultimate resistance.

No restriction is placed on the fields of application, as, in this Report, the component is treated in isolation from its use.

In every application, one design criterion is that the resistance of the part exceed the load effects. To ensure this in limit states design, the load is increased by a "partial factor on load" (load factor), and the ultimate resistance of the component is decreased by a "partial factor on resistance" (resistance factor). These factors, in combination, provide the desired reliability index for the assembly. As each application has its own load spectrum and required level of security, no general values for the partial factors are possible, and are thus not included in this Report. International Standard ISO 2394 deals with the manner in which suitable partial factors are to be determined.

Characteristic resistances obtained using this Report are, in general, the mean of test results less two standard deviations. The values may be factored to provide "safe", "working" or "rated" capacities in those applications which do not use limit states design.

Deformation and natural frequencies of components and assemblies are limited by the need to meet the dictates of the intended purpose. Such serviceability requirements are not treated in this Report.

Aluminium structures — Material and design — Ultimate limit state under static loading

1 Scope

This Technical Report provides design expressions to determine the characteristic values for the ultimate resistances of components and connections in aluminium assemblies which are subjected to known static forces.

It is intended for general applications of structural aluminium alloys other than those used in aircraft and for other special purposes. All wrought product types are included, in all thicknesses suitable for load-carrying.

2 Normative references

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

ISO 2394:1986, *General principles on reliability for structures.*

ISO 3898:1987, *Bases for design of structures — Notations — General symbols.*

ISO 8930:1987, *General principles on reliability for structures — List of equivalent terms.*

3 Symbols

The preferred symbols used in this Technical Report are based on ISO 3898 and ISO 8930, and are as follows:

a	dimension, weld throat	A	area
b	dimension		
c	distance	C	constant
d	distance, diameter		
e	eccentricity, edge distance	E	elastic modulus
f	stress		
$\bar{f}$	normalized buckling stress $= f_c/f_o$	F	action
g	fastener spacing, gap	G	shear modulus
h	web depth	H	length
i	radius of gyration	I	moment of inertia
k	factor	K	factor
		L	length
m	factor, number	M	moment
n	number	N	force
r	radius	R	resistance
s	fastener spacing, stiffener spacing		
t	thickness	T	torque
v	shear flux	V	shear force
w	width	W	section modulus
x	distance		
y	distance		
z	weld size		
α	factor, index, coefficient of thermal expansion	β	factor, angle
γ	partial factor	δ	imperfection
λ	slenderness	θ	angle
ν	Poisson's ratio	$\bar{\lambda}$	normalized slenderness $(f_o/f_e)^{1/2} = (\lambda/\pi)(f_o/E)^{1/2}$
η	ratio	ρ	density

The following suffixes are used:

b	bearing
c	compression, critical
d	design
e	elastic, Euler
f	factored, flange
g	gross
h	HAZ (heat-affected zone)
k	characteristic
l	lateral
m	maximum, material
n	net, normal
o	limiting
p	plastic, polar
r	resistance
s	shear
t	tension, torsion
u	ultimate
v	weak axis of angles, shear
w	warping, welded, web
x	axis (major)
y	yield, axis (minor)
z	axis

4 Documentation

4.1 Calculations

Calculations of the resistance of a component or connection shall include the alloy designation and temper, the guaranteed mechanical properties and any derived properties that are used, together with a complete geometric description of the component or connection, and the support conditions. Where the resistance is influenced by associated components, they shall also be fully described.

4.2 Testing

If the resistance is determined by testing, information shall be given on the method of support and load application, the number of tests, the number of parameters varied, the locations of points where strains or deflections are measured, the force/displacement relationships, and the mode of failure. Coupons shall be cut from the test specimens and the mechanical properties determined.

This information shall be sufficiently complete that a third party may interpret the results, and arrive at values for the characteristic resistances which relate to the probable strengths of the components with the same confidence as those predicted by the design expressions in this Technical Report.

5 Basic design principles

5.1 General

Components and connections are proportioned to provide a required ultimate resistance, and to perform within specified limits under service loads. All conditions arising during manufacture, transportation, assembly and construction, and the intended life in service, shall be considered in determining the suitability of the part.

5.2 Limit states

Limit states are classified as "serviceability" limit states and "ultimate" limit states.

Serviceability limit states are dictated by the function of the assembly and are treated in standards specific to the application.

Ultimate limit states are a matter of public concern. They correspond to the highest force that members, connections, assemblies or complete structures can sustain without uncontrolled distortion. The limit may be set by the need to avoid

— large deformations due to extensive yielding;

— rupture, including that due to fatigue;

— member instability;

— overall instability.

Deflection and other distortions, except insofar as they influence stability, are not considered at the ultimate limit state.

While such failure modes as overturning and foundation or anchorage failure need to be considered, they are not treated in this Technical Report, which is restricted to aluminium parts.

5.3 Design requirement

The joints, components and assemblies shall be proportioned to satisfy the inequality:

$$S_d \leqslant R_d$$

where

S_d is the design action (internal force) due to the factored load;

R_d is the design resistance (factored resistance).

5.4 Design load

Knowing the load spectrum for a particular application, characteristic loads are selected, usually based on a required return period.

The design load, F_d, is the product of the characteristic value of the load, F_k, and the partial factor on load (load factor), γ:

$$F_d = F_k \gamma$$

Values of γ are given in the applicable national standards.

The design action (internal force) is obtained from the analysis of the assembly when subjected to the design load (factored load), and is used to determine the required design resistance (factored resistance) of a component.

The design actions are expressed as follows:

N_{sd} is the design force

V_{sd} is the design shear force

M_{sd} is the design moment

T_{sd} is the design torque

5.5 Design resistance

The expressions provided in this Report give the values for the characteristic resistances, R_k.

The design resistance (factored resistance), R_d, is obtained using

$$R_d = R_k / \gamma_m$$

in which γ_m is the partial factor on resistance (resistance factor).

Values of γ_m are established for each particular application in conjunction with the partial factors on load (load factors), γ, to provide the required level of reliability. When values are not given in national standards, ISO 2394 provides guidance.

Predictors given in this Report have been targeted to give values for the characteristic resistance that are the mean value from test results less two standard deviations.

6 Basic considerations

6.1 Static actions

Characteristic values for the actions, to be used in the design, are specified in the standards appropriate to the application.

Direct actions are loads such as the weight of cargo, wind pressure and traffic. They may be static, quasi-static equivalents of impact forces, temporary, permanent, fixed, variable, planned or accidental. The type of action will determine, in part, the partial factor to be applied.

Indirect actions are those attributed to imposed changes in geometry such as are caused by expansion and settlement. In general, indirect load effects are to be avoided by using a suitable overall design, as, in normal circumstances, they are not readily predictable.

6.2 Materials

6.2.1 Aluminium alloys

In selecting an aluminium alloy for a stressed application, its suitability will be assessed on the basis of conformity to an International Standard, or European or national standard and on the following considerations, as applicable:

— strength: yield and ultimate;

— ductility: elongation and reduction in area;

— weldability and welded properties;

— corrosion resistance in the intended environment;

— formability;

— machinability;

— surface finish.

In general, extrusions will be of heat-treated alloys in T5 or higher temper, while sheet and plate may be of heat-treated or work-hardened alloys.

Where castings or forgings are to be used, there shall be close cooperation with the suppliers to establish the design properties. For foundry alloys, the values should be confirmed by tests of the finished casting.

6.2.2 Fasteners

Only bolts and solid rivets are treated in this Technical Report. Bolts may be of aluminium alloy, zinc- or cadmium-coated steel, or stainless steel. Solid rivets will preferably be of an aluminium alloy. If proprietary fasteners are used, the value of the ultimate resistance shall provide the same level of reliability as do connections proportioned according to this Report (see 6.4.2.1).

6.2.3 Welds

Welding electrodes and filler wire, and shielding gas shall be selected to suit the alloys to be joined and the method of welding (see clause 15).

6.2.4 Identification of material

All material shall be marked or stored such that the identification of the alloy and temper can be established at all stages of manufacture.

6.3 Geometrical parameters

Dimensions of profiles, members and overall assemblies shall be subject to the ruling commercial tolerances applicable to the product.

6.4 Properties

6.4.1 Physical properties

For strength design purposes, all aluminium alloys to which this Report applies shall be considered to have the following physical properties:

Elastic modulus, E	70 000 MPa
Elastic shear modulus, G	26 000 MPa
Poisson's ratio, ν	0,33
Density, ρ	2 700 kg/m^3
Coefficient of thermal expansion, α	0,000 024 per 1 °C

6.4.2 Mechanical properties

6.4.2.1 Specified properties

Yield strength, f_y, (0,2 % proof stress) is taken to be that stress in tension at which there is a 0,2 % strain offset in the stress/strain relationship.

Ultimate strength, f_u, is the highest force in tension, sustained by the test specimen, divided by the original cross-sectional area of the specimen.

Yield and ultimate strengths in tension are the basic mechanical properties specified, for each alloy and temper, in International Standards, or European or national standards. (To satisfy the requirements of the Aluminum Association, the values are expected to be exceeded in 99 % of the production, at a confidence level of 0,95.) Selected values of these strengths for some popular alloys are given in table A.1.

Yield strength in compression, yield and ultimate strengths in shear, and strength in bearing, may be established by a sufficient number of tests to provide the same level of confidence as that for the specified properties. When these values are not available, 6.4.2.2 shall be used.

6.4.2.2 Derived properties

6.4.2.2.1 Yield strength in compression

In compression, the yield strength shall be taken as the value in tension.

6.4.2.2.2 Yield and ultimate strengths in shear

For direct shear, the characteristic strengths shall be taken to be

— yield strength in shear, $f_{vy} = 0{,}6 f_y$

— ultimate strength in shear, $f_{vu} = 0{,}6 f_u$

6.4.2.2.3 Ultimate strength in bearing on fasteners

For bolts and rivets acting in bearing, the characteristic bearing strength of the connected material shall be taken to be

$$f_{bk} = 2 f_u$$

This is subject to the further restrictions given in 14.3.2.

The bearing stress on the fastener itself need not be considered.

6.4.2.3 Welded properties

Full account shall be taken of the influence of welding on the properties of aluminium, as described in 8.2.

Table A.1 gives, for some popular alloys, the values of the yield and ultimate tensile strengths of the base metal and of the metal in the heat-affected zone (HAZ) at the weld, which are acceptable for design purposes.

Table A.2 gives, for some popular alloys, values for the ultimate tensile strengths of the weld beads which are acceptable for design purposes.

Higher values may be used if they are demonstrated and can be ensured in production.

When the properties of the heat-affected zone (HAZ) are not known, they may be taken to be equal to those in the solution-treated condition for heat-treated alloys and equal to those in the annealed condition for work-hardened alloys. Because the properties of the weld bead are a function of both the base metal alloy and the filler wire alloy, any combination for which the properties are not known shall be tested to establish the strength of the weld bead itself.

7 Methods of analysis

7.1 General

For individual components and connections, the design expressions in this Report give the ultimate resistance. Behaviour may be linear or non-linear. The values obtained are not related to the method of analysis; however, as the behaviour of individual parts can influence the overall behaviour of the assembly, the method of analysis may take cognizance of the force/deformation relationships of the components.

7.2 Elastic analysis

Elastic analysis is generally considered to provide a lower bound solution, and is permitted for all assemblies, without regard for the behaviour of the individual components up to the ultimate resistance, unless geometric distortions influence the stability of the assembly.

7.3 Plastic hinge analysis

Plastic hinge analysis may be used for rigid frames in which it can be demonstrated that the required rotation at the yield hinges is available, and the destabilizing influence of compression forces is taken into account. The limited deformation capacity at bolted joints and transverse welds shall be considered when determining whether plastic hinges can be developed.

7.4 Yield line theory

Yield line theory for plates may be used, but full account shall be taken of the influence of any welds or perforations for fasteners.

7.5 Redundant lattice structures

Non-linear analysis of redundant lattice structures is permitted only where the force/deformation relationships for the components are known and can be accurately or conservatively modelled.

8 Characteristic resistance in tension

8.1 Bolted construction

8.1.1 Concentric force

For a member subjected to a concentric axial tension force, the resistance is the lesser of the values given by

$$N_k = A_g f_y$$

$$N_k = A_n k f_u$$

where

- f_y is the tensile yield strength (0,2 % proof stress) of the material;
- f_u is the ultimate tensile strength of the material;
- A_g is the area of the gross cross-section;
- A_n is the area of the net cross-section;
- k equals 1,0, unless it is demonstrated that a lower value applies for a particular alloy and temper.

8.1.2 Net sections

8.1.2.1 Where a chain of holes extends across a plate or element of a member subjected to tension, the area of the net section is given by

$$A_n = A_g - n(dt)$$

8.1.2.2 If the holes in the chain are staggered as in figure 1, the net area is given by

$$A_n = A_g - n(dt) + (n - 1)\left(s^2/4g\right)t$$

where

- A_n is the area of the net section;
- A_g is the area of the gross section;
- n is the number of holes in the chain;
- d is the hole diameter;
- t is the plate thickness;
- s is the longitudinal spacing of adjacent holes in the chain;
- g is the transverse spacing of adjacent holes.

The staggered path governs when $s^2 < 2gd$.

8.1.3 Eccentric force

8.1.3.1 General case

When axial tension force is applied with an eccentricity, e, as in figure 2, the resistance shall be the lesser of the values given by

$$N_k = f_y / (1/A_g + e/W_g)$$

$$N_k = f_u / (1/A_n + e/W_n)$$

where, in addition to the symbols in 8.1.1

W_g is the elastic section modulus of the gross section;

W_n is the elastic section modulus of the net section.

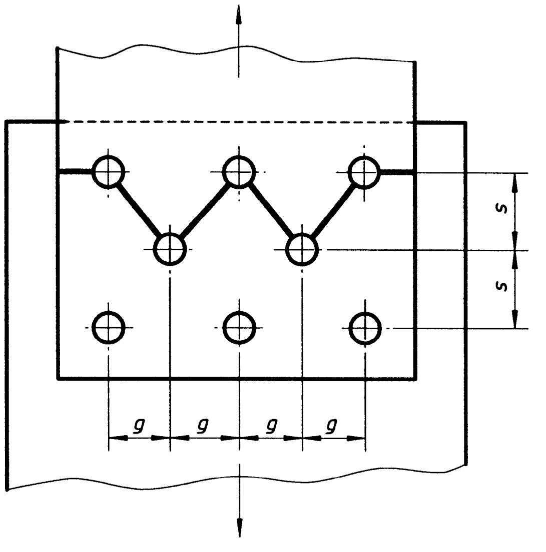


Figure 1 — Area of net section

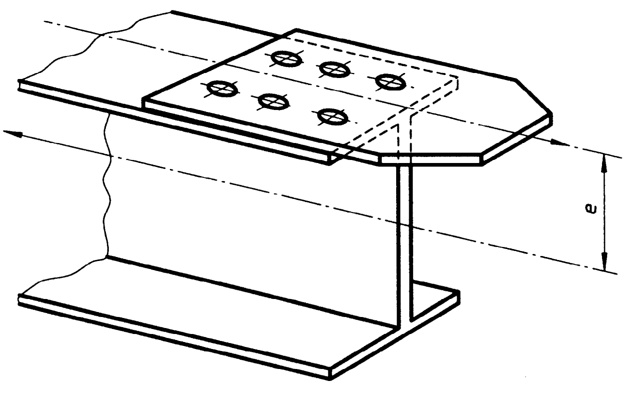


Figure 2 — Eccentric axial force

At bolted connections, the section modulus of the net section shall be calculated using

$$W_n = W_g - \Sigma(dt)_i y_i$$

where

$(dt)_i$ is the area removed by a hole at a distance y_i from the centroid of the gross cross-sectional area;

d is the hole diameter;

t is the metal thickness.

The above expressions are based on the summation of stresses derived from elastic behaviour. Full plastic analysis at the limiting condition is permitted, using the yield stress on the gross section and the ultimate stress on the net section, as is the case in the following clauses (8.1.3.2 and 8.1.4). Such an analysis shall take account of any non-symmetry in the cross-section.

8.1.3.2 Special cases

8.1.3.2.1 Eccentrically loaded gusset plates

For a plate, as shown in figure 3, loaded eccentrically in tension, the resistance shall be the lesser of the values given by

$$N_k = [(4e^2 + b^2)^{1/2} - 2e]tf_y$$

$$N_k = [(4e^2 + b^2)^{1/2} - 2e - nd]tf_u$$

where

e is the eccentricity of the force;

b is the width of the plate;

t is the thickness of the plate;

d is the hole diameter;

n is the number of holes in the transverse section.

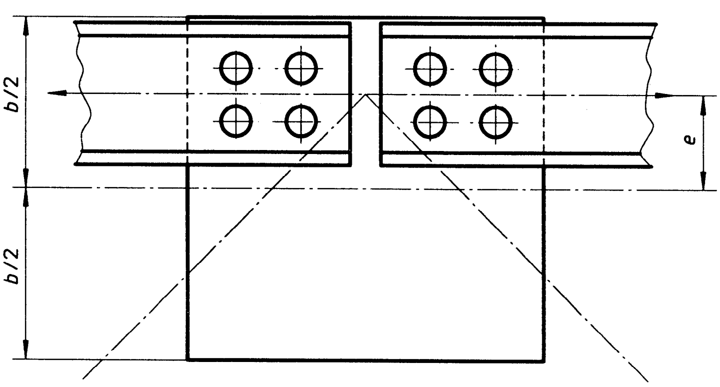


Figure 3 — Eccentrically loaded gusset plate

8.1.3.2.2 Single angles

For an isolated single angle member as in figure 4, connected at the end by a single bolt through one leg, the axial resistance in tension is the lesser of the values given by

$$N_k = (2e - d)tf_u$$

$$N_k = (b - d)tf_u$$

where

e (edge distance) $\geqslant 1{,}25d$.

If $e \geqslant b/2$ and two or more bolts are used in the connected leg, the area of the net section may be increased by one-third of the area of the outstanding leg.

The addition of lug angles (cleats, clip angles) to attach the outstanding leg is, in general, of no benefit. If any advantage is claimed, it shall be demonstrated by tests.

8.1.3.2.3 Single channels, T-sections and double angles connected eccentrically

For a single channel connected by the web, T-sections connected by the flanges, and double angles connected to one side of a gusset, the effective gross area, A'_g, to be used in 8.1.1 shall be the area of the connected element plus half the area of the outstanding elements. The effective net area, A'_n, is the effective gross area less all the holes in line across the connected elements.

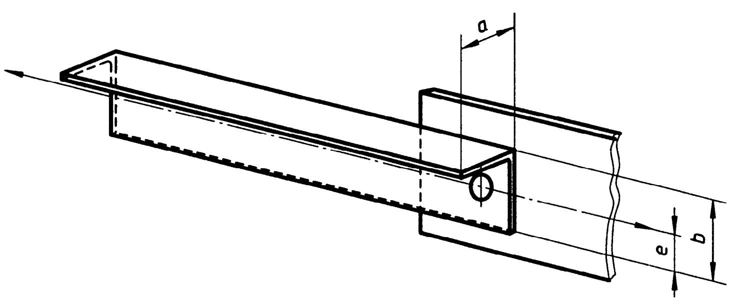


Figure 4 — Single angle connected by single bolt

8.1.4 Combined tension and bending

In the case of sections, bars, and plates, not subject to local or lateral buckling, bending in a plane of symmetry, the limiting condition is given by

$$(N_{sd}/N_d)^{\alpha} + (M_{sd}/M_d) = 1$$

where

N_{sd} is the factored axial force;

M_{sd} is the factored applied moment;

N_d $= A_g f_y/\gamma$ and $M_d = W_p f_y/\gamma$ or

N_d $= A_n f_u/\gamma$ and $M_d = W_n f_u/\gamma$

A_g is the gross area;

A_n is the net area;

W_p is the plastic section modulus of the gross section;

W_n is the plastic section modulus of the net section;

f_y is the yield strength;

f_u is the ultimate strength;

α = 2 for solid sections, T, C, Z, H, and similar sections bending about the weak axis,

= 1,4 for hollow sections and C, Z, H, and similar sections bending about the strong axis;

γ is the resistance factor.

8.2 Full penetration butt welded construction

8.2.1 Full width transverse welds

For full penetration, full width, transverse butt welded joints in tension, the tensile resistance is the least of the values given by the expressions:

$$N_k = tLf_y$$

$$N_k = tLf_{hu}$$

$$N_k = tLf_{wu}$$

where

t is the plate thickness;

L is the length of the butt weld;

f_y is the yield strength of the unaffected base metal;

f_{hu} is the ultimate tensile strength in the HAZ (table A.1 gives some values);

f_{wu} is the ultimate tensile strength in the weld bead (table A.2 gives some values).

Partial penetration butt welds are not recommended for engineered designs but, should the need arise, then t in 8.2.1 shall be replaced by the total effective throat.

8.2.2 Full width oblique welds

For a full width oblique weld, as shown in figure 5, if the angle θ between the direction of the weld and the direction normal to the applied stress is less than 45°, the dimension L shall be taken as the width of the welded element measured in a direction normal to that of the applied stress. For angles θ greater than 45°, the treatment for longitudinal welds (8.2.3) shall be used except that the resistance shall not exceed

$$N_k = tLf_{wu}/\sin 2\theta$$

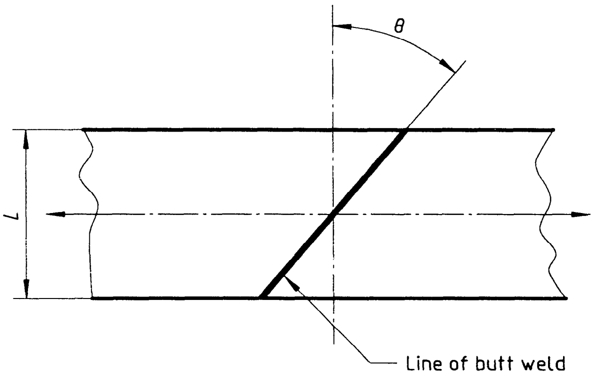


Figure 5 — Oblique butt weld in tension

8.2.3 Longitudinal welds

For members with longitudinal welds, and in other cases where only a portion of the cross-section is influenced by welding, the effective transverse section is obtained by reducing the thickness in the HAZ (15.2) to an effective thickness, t', given by

$$t' = t(f_{hy}/f_y)$$

where

t is the original thickness;

f_y is the tensile yield strength in the unaffected base metal;

f_{hy} is the yield strength in the HAZ (table A.1).

9 Characteristic resistance in compression

9.1 Concentric force

For a member subjected to a concentric axial compressive force, the resistance is given by

$$N_k = A_g f_c$$

where

A_g is the gross area;

f_c is the buckling stress determined using 9.4.

9.2 Limiting stress

The maximum compressive stress at any point in a cross-section shall not exceed the limiting stress, f_o, appropriate to the situation, as follows:

a) the yield strength of the base metal, f_y;

b) the yield strength of the HAZ at a transverse weld away from lateral supports, f_{hy} (15.8.2);

c) the ultimate tensile strength of the HAZ at a transverse weld adjacent to a lateral support, f_{wu};

d) the weighted average yield strength in a longitudinally welded member, f_o (9.5.2);

e) the initial local buckling stress in an outstanding flange or other element where local buckling precipitates collapse, f_c (9.5.3);

f) the mean post-buckling stress where local buckling does not lead to immediate collapse, $(f_y f_c)^{1/2}$ (9.5.4);

g) the buckling stress of a chord in a lattice column, f_c (9.5.1).

9.3 Normalized slenderness

9.3.1 The normalized slenderness of a component which may buckle under the action of compressive stress is given by

$$\bar{\lambda} = (f_o/f_e)^{1/2} = (\lambda/\pi)(f_o/E)^{1/2}$$

where

$\bar{\lambda}$ is the normalized slenderness;

f_o is the limiting stress in compression;

f_e is the elastic buckling stress equal to $\pi^2 E/\lambda^2$

E is the elastic modulus;

λ is the slenderness appropriate to the mode of buckling.

9.3.2 In other cases of buckling, the normalized slenderness may be expressed as

Columns

$$\bar{\lambda} = (N_o/N_e)^{1/2}$$

Beams

$$\bar{\lambda} = (M_o/M_e)^{1/2}$$

where

N_o is the limiting axial force when $\lambda = 0$;

M_o is the limiting moment when $\lambda = 0$;

N_e is the theoretical axial force to cause elastic buckling;

M_e is the theoretical moment to cause elastic buckling.

The value of $\bar{\lambda}$ shall be used in 9.4 to determine the normalized buckling stress, force or moment.

9.4 Normalized buckling stress

The highest mean axial stress sustained before instability occurs is the buckling stress. The ratio of the buckling stress, f_c, to the limiting stress, f_o, is the normalized buckling stress

$$\bar{f} = f_c/f_o$$

The relationship between the normalized buckling stress, $\bar{f}$, and the normalized slenderness, $\bar{\lambda}$, is given by

$$\bar{f} = \beta - \left(\beta^2 - 1/\bar{\lambda}^2\right)^{1/2}$$

where

$$\beta = [1 + \alpha(\bar{\lambda} - \bar{\lambda}_o) + \bar{\lambda}^2]/2\lambda^2$$

$\bar{\lambda}_o$ is the value of the normalized slenderness below which the limiting stress can be sustained, and will vary between different alloys, members and elements.

To reflect the different stress/strain relationships for work-hardened and heat-treated alloys, different values of α are used. Values of α may also be varied for columns, beams and plates, different shapes, and for symmetrical and asymmetrical profiles.

A review of suitable values for α and λ_o is given in annex A, together with some representative curves of the relationships.

If forces or moments are used rather than stress, the value of $\bar{f}$ given by the above expression represents the normalized buckling force, N_c/N_o, or the normalized buckling moment, M_c/M_o.

9.5 Flexural buckling of columns

9.5.1 General case

The slenderness of a column failing in pure flexure is given by

$$\lambda = KL/i$$

where

- L is the length between lateral restraints;
- i is the radius of gyration for the axis of flexure;
- K is the effective length factor.

This slenderness, in conjunction with the appropriate limiting stress, f_o (9.2), gives the normalized slenderness, $\bar{\lambda}$ (9.3). The normalized buckling stress, $\bar{f}$, shall then be obtained using 9.4.

The axial resistance of a compression member is given by

$$N_k = (\bar{f}f_o)A_g = f_cA_g$$

where

- f_o is the limiting stress;
- f_c is the buckling stress;
- A_g is the gross cross-sectional area of the column.

9.5.2 Influence of longitudinal welds

For compression members with longitudinal welds only, the limiting stress f_o, is given by

$$f_o = f_y - (A_h/A_g)(f_y - f_{hy})$$

where

f_y is the yield strength of the base metal;

f_{hy} is the yield strength of the HAZ;

A_g is the area of the gross cross-section;

A_h is the area of the cross-section of the HAZ's (15.2).

This limiting stress shall be used in 9.3 to obtain $\bar{\lambda}$, which is then used in 9.4 to obtain $\bar{f}$.

The axial resistance of the compression member is then given by

$$N_k = k(\bar{f} f_o) A_g$$

where

$$k = 0{,}9 + |1 - \bar{\lambda}|\ 0{,}1 \leqslant 1{,}0$$

($|1 - \bar{\lambda}|$ is the absolute value.)

9.5.3 Influence of local buckling that leads to failure

For pin-ended columns, the local buckling of outstanding flanges in I, Z and C sections, and of the walls of cylindrical tubes, shall be deemed to limit the capacity. The local buckling stress, f_c, is obtained using 12.1 for flat elements, and 12.5 for curved walls.

This local buckling stress shall be taken as the limiting stress, f_o, in conjunction with the overall slenderness to calculate the normalized slenderness, $\bar{\lambda}$, in 9.3 for use in 9.4 to give the resistance of the column.

9.5.4 Influence of local buckling with post-buckling strength

Flat elements with both long edges supported can sustain a mean compressive stress in excess of that causing initial local buckling. The buckling stress, f_c, is obtained from 12.1. The limiting stress, f_o, is then given by

$$f_o = (f_y f_c)^{1/2} = \bar{f}^{1/2} f_y$$

where

$\bar{f}$ is the normalized buckling stress.

This value of f_o shall be used in 9.3 and 9.4 to give the axial resistance of the column.

9.6 Torsional and torsional-flexural buckling of columns

9.6.1 Open sections symmetrical about one axis

In open sections with only one axis of symmetry, torsional buckling combines with flexure about the axis of symmetry. The slenderness to be used in 9.3 to obtain the normalized slenderness is given by

$$\lambda = [1 + 0{,}4(x_o/i_p)(\lambda_2/\lambda_1)^2]\lambda_1$$

where

λ_1 is the larger of λ_t and λ_f;

λ_2 is the smaller of λ_t and λ_f;

λ_f is the slenderness for flexural buckling about the axis of symmetry

$= KL/i_x$,

λ_t is the slenderness for torsional buckling

$= 5(I_p/I_t)^{1/2}$ or as modified by 9.6.3;

x_o is the distance from the centroid to the shear centre of the cross-section;

I_p is the polar moment of inertia about the shear centre

$= I_x + I_y + Ax_o^2 = Ai_p^2$

I_t is the St. Venant torsion constant;

i_p is the polar radius of gyration about the shear centre;

i_x is the radius of gyration for the axis of symmetry;

A is the cross-sectional area;

L is the length.

Calculation of the slenderness, λ_f, for built-up sections such as double angles, shall include the influence of shear according to 9.7.

9.6.2 Angles, T-sections and cruciform sections

In the case of single or multiple angles, T-sections and cruciform sections, all with equal legs, the slenderness for torsional buckling is given by

$$\lambda = 5b/t$$

where

b is the leg length;

t is the leg thickness.

For extruded sections, b shall be measured from the start of the root fillet radius. For shapes with no thickening at the root (e.g. cold-formed shapes), b shall be measured from the median line of the adjacent leg.

9.6.3 Influence of warping resistance

In open sections, such as channel or hat shapes, the resistance to torsional buckling is increased by the warping resistance of the section. The slenderness for torsional buckling of sections with one axis of symmetry is given by

$$\lambda_t = 5(I_p/I_t)^{1/2}/\left(1 + 25C_w/I_tL^2\right)^{1/2}$$

where

C_w is the warping constant;

L is the length between restraints.

This value of λ_t shall be used in 9.6.1.

9.6.4 Asymmetrical open sections

In the case of open sections with no axis of symmetry, flexure about both principal axes combines with torsion to give the buckling mode. If a design is required, reference shall be made to the literature to obtain the elastic buckling stress, f_e, for use in 9.3.

9.7 Built-up compression members

The resistance of built-up compression members composed of multiple bars, such as double angles and battened channels, connected together by widely spaced fasteners or welds, is determined in accordance with 9.5, using a slenderness given by

$$\lambda = \left(\lambda_1^2 + \lambda_2^2\right)^{1/2}$$

where

λ_1 is the overall slenderness for the built-up section treated as a whole, for bending about the built-up axis;

λ_2 is the slenderness of the individual bars between points of interconnection for bending about an axis parallel to that used to compute λ_1.

The connections between the bars, at each location, shall be designed to resist a total shear force equal to 2,5 % of the total axial force. There shall be rigid connections between the bars at each end, and, preferably, there should be not less than two intermediate connections.

9.8 Lattice columns

To determine the axial compressive force to cause overall buckling of lattice columns, the limiting stress, f_o, shall be the buckling stress for an individual chord member, f_c. Design shall then be in accordance with 9.4. The influence of shear flexibility in lattice columns (λ_2 in 9.7) may be neglected.

The planes of diagonal bracing shall be designed to resist a total shear force across the mast equal to 2,5 % of the total axial force.

10 Bending

10.1 Moment resistance

The moment resistance of a member shall be given by the appropriate expression from the following cases:

10.1.1 For failure in the plane of bending

a) Fully plastic: $M_k = W_p f_y$

With longitudinal welds (8.2.3): $M_k = W_{ep} f_y$

b) Limited by first yield: $M_k = W f_y$

With longitudinal welds (8.2.3): $M_k = W_e f_y$

c) Limited by the ultimate strength in tension at connections:

Bolted: $M_k = W_n f_u$

Welded transversely: $M_k = W_p f_{wu}$

d) Limited by local buckling of outstanding flanges which leads to collapse [12.1.3.2]: $M_k = Wf_c$

e) Limited by local buckling of elements with post-buckling reserve [12.2.2.2]: $M_k = W_e f_y$

f) Limited by chord buckling in lattice beams (9.5): $M_k = Adf_c$

10.1.2 For failure by lateral-torsional buckling (10.2) $M_k = Wf_{cl}$

10.1.3 In 10.1.1 and 10.1.2, the symbols are as follows:

M_k is the characteristic moment resistance;

A is the total cross-sectional area of the compression chords;

d is the depth of lattice beam;

W is the elastic section modulus of the gross section;

W_e is the elastic section modulus of the effective section (12.2.2 for local buckling, 8.2.3 for longitudinal welds);

W_{ep} is the plastic section modulus for the effective section (8.2.3 for welded sections);

W_n is the section modulus of the net section (8.1.3.1);

W_p is the plastic section modulus of the gross section;

f_y is the yield strength of the base metal;

f_u is the ultimate tensile strength of the base metal;

f_{wu} is the ultimate tensile strength at the weld;

f_{hy} is the yield strength of the HAZ;

f_c is the critical stress for buckling of chords (9.5) or of outstanding flanges, which leads to failure (9.5.3);

f_{cl} is the critical stress for lateral-torsional buckling (10.2).

The fully plastic condition, given by 10.1.1 a), shall only be used where there is a demonstrable security against overall and local instability. In general, for cases controlled by yielding, the value obtained from 10.1.1 b) shall be used as the limiting condition.

10.2 Lateral-torsional buckling

10.2.1 For uniform moment, and where the maximum moment occurs in the span, the slenderness for lateral-torsional buckling, is given as follows:

a) For beams between points of lateral restraint of the compression flange or points of torsional restraint:

$$\lambda = (W_x L)^{1/2} / [I_y \left(0{,}04 I_t + C_w / (KL)^2\right)]^{1/4}$$

b) For beams with continuous lateral restraint to the tension flange and points of lateral restraint to the compression flange:

$$\lambda = (W_x L)^{1/2} / [I_y + \left(0{,}04 I_t + C_w / (KL)^2\right)]^{1/2}$$

In both a) and b),

W_x is the elastic section modulus about the axis of bending (X-axis);

I_y is the moment of inertia about the weak axis (Y-axis);

L is the distance between points of lateral restraint to the compression flange or to twisting;

I_t is the St. Venant torsion constant;

C_w is the warping constant;

K is a factor representing the restraint to warping at the ends, usually taken as 1.

The limiting stress, f_o, shall be one of those described in 9.2. The normalized slenderness (9.3) shall be used with the appropriate buckling curve for columns (9.4) to give the normalized buckling stress, $\bar{f}$, and the actual buckling moment:

$$M_k = (\bar{f}f_o)W$$

where W is the section modulus.

10.2.2 Linear moment gradients, with the maximum moment at an end, shall be treated as an equivalent uniform moment given as follows:

$$0{,}6M_1 + 0{,}4M_2 \geqslant 0{,}4M_1,$$

where the end moments are M_1 and M_2, $|M_1| > |M_2|$, and M_2 is negative if it creates double curvature.

The maximum end moment is subject to the limits of 10.1.1.

11 Beam-columns

11.1 Moment with axial compressive force

For members subjected to combined axial and bending forces, the ultimate limit state shall be given by the more restrictive of the conditions in 11.1.1 and 11.1.2.

11.1.1 Failure in the plane of bending (about the X-axis)

a) Limited by yielding in compression:

$$N_{sd}/N_{dx} + M_{sd}/M_d(1 - N_{sd}/N_{ex}) = 1$$

b) Limited by an elastic extreme fibre compressive stress, such as local buckling in flanges and chord buckling in latticed masts:

$$N_{sd}/A + M_{sd}/W_x(1 - N_{sd}/N_{ex}) = f_o/\gamma_m$$

c) Limited by an extreme fibre tension stress:

$$M_{sd}/W_x(1 - N_{sd}/N_{ex}) - N_{sd}/A = f_o/\gamma_m$$

11.1.2 Failure by lateral-torsional buckling

$$N_{sd}/N_{dy} + M_{sd}/M_{dl}(1 - N_{sd}/N_{ex}) = 1$$

11.1.3 In 11.1.1 and 11.1.2, the symbols are as follows:

A is the gross cross-sectional area;

W_x is the elastic section modulus applicable to the fibre with the maximum compressive stress for case 11.1.1 b), and with the maximum tensile stress for case 11.1.1 c);

N_{sd} is the design applied axial force;

N_{dx} is the design axial resistance for buckling in the plane of bending, X-axis, (9.5);

N_{dy} is the design axial resistance for buckling about the Y-axis, i.e. out of the plane of bending, (9.5);

N_{ex} is the theoretical axial force for elastic flexural buckling in the plane of bending and is equal to $\pi^2 EI_x/(KL)^2$;

M_{sd} is the design applied bending moment;

M_{dl} is the design moment resistance limited by lateral-torsional buckling (10.2);

M_d is the design moment resistance limited by yielding;

f_o is the limiting stress at the extreme compression or tension fibre (9.2);

γ_m is the partial factor on resistance.

If extensive plastification can take place in short members such as is permitted for combined tension and bending (8.1.4), a limiting combination demonstrated by test or by computer simulation may be used. The value of the compressive force, N, shall not exceed that given by 9.5 for overall flexural buckling about the Y-axis.

11.1.4 For axially loaded members subjected to moments about both principal axes, with the maximum moment in the span, in which torsion is not a consideration, and the most highly stressed fibre is the same for M_x and M_y, the ultimate limit state is given as follows:

a) Limited by yielding in compression:

$$N_{sd}/N_d + M_{sdx}/M_{dx}(1 - N_{sd}/N_{ex}) + M_{sdy}/M_{dy}(1 - N_{sd}/N_{ey}) = 1$$

b) Limited by an elastic extreme fibre stress:

$$N_{sd}/A + M_{sdx}/W_x(1 - N_{sd}/N_{ex}) + M_{sdy}/W_y(1 - N_{sd}/N_{ey}) = f_o/\gamma_m$$

In both a) and b),

W_x, W_y are the elastic section moduli about the X- and Y-axes;

N_{sd} is the design applied axial force;

N_d is the design axial resistance;

N_{ex}, N_{ey} are the theoretical axial forces for elastic buckling about the X- and Y-axes;

M_{dx}, M_{dy} are the design moment resistances for bending about the X- and Y-axes;

M_{sdx}, M_{sdy} are the design applied moments about the X- and Y-axes;

f_o is the limiting stress from 9.2;

γ_m is the partial factor on resistance.

Torsional-flexural buckling under the action of axial force and biaxial moments shall be treated by a rational method which includes the influence of the axial force on the torsional as well as on the flexural components of the buckled mode.

11.2 Eccentrically loaded columns

11.2.1 General case

Eccentrically loaded columns shall be proportioned to satisfy 11.1.

11.2.1.1 For failure in the plane of bending:

a) When limited by yielding in compression, 11.1.1 a) shall apply, using a design applied moment given by

$$M_{sd} = N_{sd}e$$

b) When limited by elastic compressive stress, such as for local buckling and for chords in lattice columns, 11.1.1 b) shall apply, using an applied moment given by

$$M_{sd} = 1{,}2N_{sd}e$$

c) When limited by tension stress, 11.1.1 c) shall apply, using a moment given by

$$M_{sd} = 1{,}2N_{sd}e$$

11.2.1.2 For failure by lateral-torsional buckling, 11.1.2 shall apply, using a moment given by

$$M_{sd} = N_{sd}e$$

where

N_{sd} is the design applied axial force;

e is the eccentricity.

11.2.2 Single angle struts

Discontinuous single angle compression members, loaded through one leg, as for internal bracing members, fail in a flexural-torsional mode. The slenderness used to calculate $\bar{\lambda}$, for direct entry into 9.4 to give the mean axial stress at failure, shall be given by

$$\lambda = \left(\lambda_t^2 + \lambda_v^2\right)^{1/2}$$

where

λ_t $= 5b/t$ (See 9.6.2.)

λ_v $= KL/i_v$

b, t are the larger leg width and thickness;

L is the length;

i_v is the radius of gyration about the minimum axis;

K is the effective length factor

= 0,8 for 1 bolt connection,

= 0,7 for 2 or more bolt connections.

The mean axial stress shall not exceed $0{,}5f_y$ for 1 bolt connections, or $0{,}67f_y$ for 2 or more bolt connections or welded connections.

11.3 Shear force in beam-columns and eccentric columns

a) The maximum factored shear force, V_{max}, in beam-columns is given by

$$V_{max} = V_{sd}/(1 - N_{sd}/N_e) \geqslant N_{sd}/40$$

b) The maximum shear force, V_{max}, in an eccentrically loaded column is given by

$$V_{max} = 4N_{sd}e/L(N_e/N_{sd} - 1) \geqslant N_{sd}/40$$

In both a) and b),

V_{sd} is the design shear force due to the lateral load;

N_{sd} is the design axial force;

N_e is the theoretical load to cause elastic column buckling in the plane of the applied shear force $= \pi^2 EI/(KL)^2$

I is the moment of inertia for the axis of bending;

L is the column length;

K is the effective length factor;

e is the eccentricity.

12 Local buckling

12.1 Flat elements in compression

12.1.1 Slenderness

Local buckling takes the form of waves in the elements of a section. The wavelength and critical stress are functions of the cross-sectional dimensions, and are taken to be independent of the overall length of the element. The slenderness is given by

$$\lambda = mb/t$$

where

m is a coefficient, particular to the geometry, support conditions, and stress distribution, determined using 12.1.2 *et seq*;

b is a ruling dimension of the cross-section;

t is the thickness of the element.

The slenderness λ shall be used in 9.3 to obtain the normalized slenderness $\bar{\lambda} = (\lambda/\pi)(f_y/E)^{1/2}$. Clause 9.4 shall then be used to give the stress, f_c, at initial buckling.

The width of an element shall be measured from the intersections of the median lines of adjacent elements. Corner radii of fillets or forming shall be neglected, unless demonstrated by test or computer analysis that a higher buckling stress can be carried.

This clause deals with local buckling. For torsional buckling, root fillets in extrusions contribute to the torsion constant (9.6.2).

12.1.2 Uniform axial compression

12.1.2.1 For a flat element of width b, thickness t_1, with both long edges supported by adjacent elements of width a, thickness t_2, which are also supported at both long edges, as shown in figure 6, forming part of a uniformly stressed section, the value of m is given by

$$m = 1{,}2 + 0{,}4k \leqslant 1{,}6$$

where

$$k = (a/t_2)/(b/t_1) < 1$$

If $k > 1$, interchange elements a and b.

12.1.2.2 For a flat element of width b, free on one long edge and supported at the other long edge by an adjacent element of width a, which is supported on both long edges (e.g. supported by a web as in figure 7), the value of m is given as follows:

a) for $k < 3$

$$m = 3 + 0{,}6k \leqslant 5$$

b) for $k > 3$

$$m = 5$$

and the web is checked using $\lambda = 1{,}6a/t_2$.

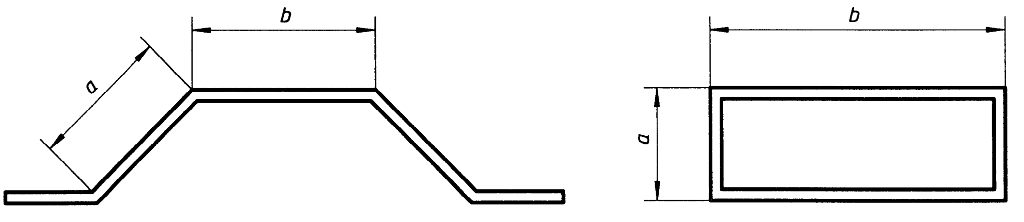


Figure 6 — Flat elements supported along two edges

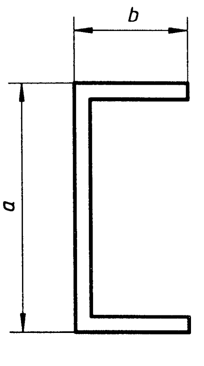


Figure 7 — Flat element of width *b* supported at one long edge

12.1.3 Uniform compression due to bending

12.1.3.1 For a flat element of width b, thickness t_1, supported at both long edges by webs of width a, thickness t_2, in uniform compression caused by overall bending of the member (figure 6), the value of m is given as follows:

a) for $k < 2{,}5$

$$m = 1{,}2 + 0{,}16k \leqslant 1{,}6$$

b) for $k > 2{,}5$

$$m = 1{,}6$$

and the web is checked using 12.1.4.

12.1.3.2 For a flat element of width b, thickness t_1, supported at one long edge by a web of width a, thickness t_2, and free at the other long edge, in uniform compression caused by overall bending of the member (figure 7), the value of m is given as follows:

a) for $k < 8$

$$m = 3 + 0{,}25k \leqslant 5$$

b) for $k > 8$

$$m = 5$$

and the web is checked using 12.1.4.

In both 12.1.3.1 and 12.1.3.2,

$$k = (a/t_2)/(b/t_1)$$

12.1.4 Non-uniform stress distribution

12.1.4.1 For a flat element, supported on two long edges, carrying a stress varying linearly across the element from f_1 to f_2, (e.g. the web of a beam), the value of m is given as follows:

a) for $-1 < f_2/f_1 < 1$

$$m = 1{,}1 + 0{,}5f_2/f_1$$

b) for $f_2/f_1 < -1$

$$m = 1{,}2/(1 - f_2/f_1)$$

12.1.4.2 For a flat element, supported at one edge and free at the other, with a stress varying linearly across the element (e.g. an angle bending about the weak axis), the value of m is given as follows:

a) When the maximum compressive stress is at the free edge:

$$m = 2{,}5(3 + f_2/f_1)^{1/2}$$

If $|f_2/f_1| > 3$, elastic buckling does not occur.

b) When the maximum compressive stress is at the supported edge:

$$m = 2{,}5(1 + 3f_2/f_1)^{1/2}$$

If $|f_2/f_1| > 1/3$, elastic buckling does not occur.

In both a) and b),

f_1 is the maximum compressive stress (negative),

f_2 is the stress at the other edge (positive if tension).

Initial buckling in this case precipitates failure.

12.1.4.3 In the case of I- and C-sections bending about the weak axis, the compressive stress at the extreme fibre of the flange shall be limited to a value obtained using:

$$m = 2{,}6 + 0{,}19k \leqslant 4$$

where

b, t_1 are the outstanding width and thickness of the flange;

a, t_2 are the depth and thickness of the web;

k $= (a/t_2)/(b/t_1)$

12.1.5 Sandwich panel skins

For the skin of a sandwich panel, bonded to a flexible core, the buckling stress shall be obtained from the formula in 9.4 for plates, using the yield stress for the alloy and the value of the slenderness given by

$$\lambda = 4{,}5[E/(E_c G_c)^{1/2}]^{1/3}$$

where

E is the elastic modulus of the skin;

E_c is the elastic modulus of the core normal to the skin;

G_c is the shear modulus of the core in a plane normal to the skin and to the direction of the stress.

The same value may be used for the compression component of an applied shear stress.

There is no post-buckling strength unless the skin is attached to the edge-framing in which case a special study in required.

12.2 Post-buckling strength of flat elements in compression

12.2.1 Outstanding flanges

12.2.1.1 In columns and beams subject to overall buckling, the local buckling of an outstanding flange shall be assumed to precipitate collapse.

12.2.1.2 Should an I-section subjected to bending be fully restrained against lateral movement there will be a reserve of strength in the flange after initial local buckling, as the material adjacent to the flange/web junction can now accept increasing stress. An effective flange thickness may be used, assumed to be capable of sustaining the yield stress. The effective thickness, t', of the flange shall be given by

$$t' = t(f_c/f_y)^{1/2}$$

where

t is the original thickness of the flange;

f_c is the initial local buckling stress for the flange [12.1.1 and 12.1.2.2].

12.2.2 Flat elements supported on both long edges

For any element with both long edges supported, subjected to any distribution of compressive stress, there will be a reserve of post-buckling strength. The procedures to take this strength into account shall be as given in 12.2.2.1 and 12.2.2.2.

12.2.2.1 Columns and beams subject to overall buckling

To determine the resistance to overall buckling, the limiting stress, f_o, to be used in 9.3 and 9.4 shall be given by

$$f_o = (f_c/f_y)^{1/2}$$

where f_c is the compressive stress at an extreme fibre that causes local buckling (12.1.2.1, 12.1.3.1, 12.1.4.1).

12.2.2.2 Members subjected to bending, with no overall buckling

For each element of the section where local buckling occurs prior to yielding, the thickness shall be reduced to

$$t' = t(f_c/f_y)^{1/2}$$

where

t is the original thickness;

f_c is the maximum compressive stress to cause buckling for the particular element and stress distribution (12.1.2.1, 12.1.3.1, 12.1.4.1);

f_y is the yield stress.

The section modulus of the effective section shall then be used, with the yield strength as the limiting stress, to give the bending resistance.

12.3 Elements with stiffeners

12.3.1 Lipped flanges

12.3.1.1 The local buckling stress of the flange of a channel or I-section which incorporates a lip at the unsupported edge [see figure 8 a)] (sometimes referred to as "distortional" buckling) shall be determined using the slenderness:

$$\lambda = \pi[EI_p/\left(GI_t + 2(EC_w k)^{1/2}\right)]^{1/2}$$

where

E, G are the elastic and shear moduli, respectively;

I_p is the polar moment of inertia for the combined lip and flange about the flange-web connection;

I_t is the torsion constant for the combined lip and flange;

C_w is the warping constant for the combined lip and flange rotating about the flange-web connection $= I_s b^2$

I_s is the moment of inertia of the stiffener about the inside surface of the flange;

k is $Et^3/5(a + 0{,}5b)$

a is the web depth;

b is the flange width;

t is the web thickness.

The dimensions a, b, c shall be measured to the median lines of the adjoining elements.

12.3.1.2 For a channel shape of uniform thickness, with simple lips on the flanges as shown in figure 8 b), the following expression is used:

$$\lambda = \frac{5b}{t}\left(\frac{1 + 3\beta}{1 + \beta + 3{,}7\left(\frac{\beta^3 (b/t)^2 + 0{,}1}{\alpha + 0{,}5}\right)^{1/2}}\right)^{1/2}$$

where

a is the web depth;

b is the flange width;

c is the lip width;

α $= a/b$

β $= c/b$

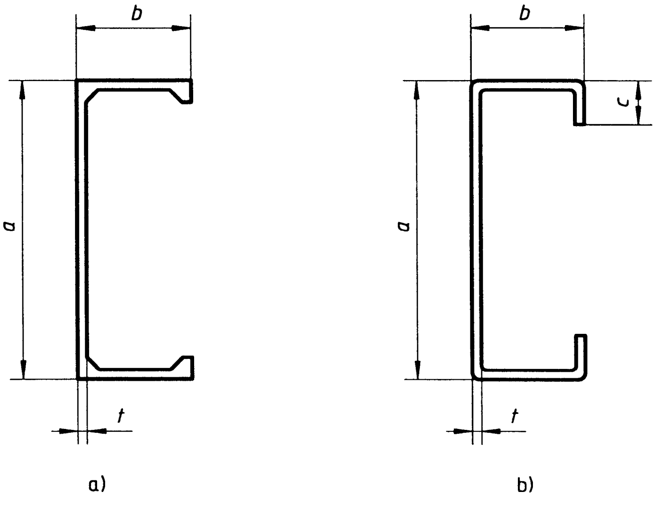


Figure 8 — Lipped flanges

12.3.2 Stiffened panels

For panels with multiple stiffeners, loaded in the direction of the stiffeners, the slenderness is given by

$$\lambda = 0{,}7\beta b/i \leqslant a/i$$

where

b is the panel width;

a is the panel length;

i is the radius of gyration of the stiffened cross-section;

β $= 1{,}8\left(I/t^3\right)^{1/4}$ for sheet with added stiffeners,

$= 1{,}8(\eta i/t)^{1/2}$ for formed sheet;

η is the ratio of developed to net width of formed sheet;

I is the moment of inertia per unit width of the stiffened section.

12.4 Flat elements in shear

12.4.1 Buckling stress

For a panel $a \times b$, where a is the larger dimension, subjected to a uniform shear stress, the normalized slenderness is given by

$$\bar{\lambda} = (f_{sy}/f_{se})^{1/2} = 0{,}45[f_{sy}/\left(1 + 0{,}75k^2\right)E]^{1/2}(b/t)$$

where

f_{sy} is the shear yield strength

$= 0{,}6f_y$,

f_{se} is the critical shear stress for initial elastic buckling

$= 5E\left(1 + 0{,}75k^2\right)(t/b)^2$

k $= b/a$

t is the panel thickness.

This value of $\bar{\lambda}$ shall be used in 9.4, with the appropriate values of α and $\bar{\lambda}_o$ for plates in the relevant alloy, and with the shear yield strength, f_{sy}, as f_o, to obtain $\bar{f}$. The initial buckling stress in shear, f_{sc}, is then given by

$$f_{sc} = \bar{f} f_{sy}$$

12.4.2 Post-buckling strength of shear webs

a) For a web with flanges, and stiffeners at the supported and loaded points, with or without other intermediate stiffeners, the shear resistance of the web, with no assistance from the flanges, is given by

$$V_{kw} = [(2f_{sc}v_k/t)^{1/2} - f_{sc}]ht$$

b) Additional shear resistance due to the presence of the flanges is given by

$$V_{kf} = 2(M_p v_k)^{1/2}$$

c) The total shear resistance is then given by

$$V_k = V_{kw} + V_{kf} \leqslant (hv_k + 4M_p/s)$$

The contribution by the flanges shall not be included in the strength of riveted or welded panels in which the shear resistance at the boundary, v_k, is less than $f_{sy}t$, where f_{sy} is the shear yield strength of the web base metal.

In a), b) and c),

h is the depth of web;

s is the stiffener spacing;

f_{sc} is the shear stress to cause initial buckling (12.4.1);

M_p is the fully plastic moment of the flange section for bending in the plane of the web, with no web material included: for simple flanges in welded plate girders this is given by

$$M_p = \left(w_f t_f^2/4\right) f_y$$

where

w_f, t_f are the width and thickness of the flange;

f_y is the yield strength of the flange alloy.

If the vertical web stiffeners are welded to the flanges, the value of f_y is replaced by f_{hy} for the HAZ of the flange alloy.

If the flange carries an axial force, N, M_p is replaced by

$$M'_p = M_p[1 - (N/N_p)^2]$$

where

N_p $= A_f f_y$ or $A_f f_{hy}$, as applicable;

A_f is the flange area;

v_k is the shear resistance per unit length of the web to flange connection, or of any seams in the web; the value to be used shall be the least of those given by

$0{,}6tf_y$, $0{,}6tf_{hu}$, $2(0{,}6af_{wu})$ and R_k/g

where

a is the throat of the fillet weld;

t is the web thickness;

f_{hu} is the ultimate tensile strength in the HAZ;

f_{wu} is the ultimate tensile strength of the weld metal in the fillet welds;

f_y is the tensile yield strength of the base metal;

R_k is the resistance of each fastener between the web and the flange or in the web seams;

g is the fastener spacing.

12.4.3 Webs with transverse and longitudinal stiffeners

Where longitudinal and transverse stiffeners divide the web into panel areas $a \times b$, $(a > b)$, the buckling stress, f_{sc}, shall be determined for the panel which gives the lowest value, at the section of interest. This value shall be used in 12.4.2 to obtain the total shear resistance, with h as the full web depth.

12.4.4 Combined shear and moment in webs

For combined shear force and moment, if the web has buckled in shear, the web shall be assumed to make no contribution to the overall moment of resistance, which will be provided by the flanges only.

12.4.5 Shear web stiffeners

12.4.5.1 Transverse stiffeners

Transverse stiffeners shall be proportioned to carry the higher of the shear forces in the panels each side of the stiffener. The stiffener shall be treated as a strut of length equal to the web depth. The moment of inertia of a double stiffener section shall be calculated about a centroid in the plane of the web.

Stiffeners on one side only shall be considered to be composed of the stiffening element and a width of the web given by

$$2(E/f_y)^{1/2}t$$

and shall be treated as eccentrically loaded.

If the post-buckling strength is to be used, the attachments to the web, in each of the upper and lower halves of the stiffener, shall be sufficient to transmit the design shear force.

12.4.5.2 Longitudinal stiffeners

Longitudinal stiffeners shall be designed as struts, of length s, to carry an axial force given by

$$N = (f_{sc}v_k t)^{1/2}s$$

but not less than the compression force caused by overall bending,

where

s is the length of stiffener (distance between transverse stiffeners);

f_{sc} is the initial shear buckling stress for the wider panel adjacent to the stiffener;

v_k is the shear resistance per unit length (12.4.2).

12.4.6 Web crushing

Unstiffened webs carrying local forces, such as at supports, may be subjected to the combined action of bending and transverse compressive stresses. The compressive resistance, N_k, acting in the plane of the web is given as follows:

a) for flat webs

$$N_k = k(n + h)tf'_c \leqslant ntf_y$$

b) for webs with a bend radius at the corner

$$N_k = k(11 + 0{,}07n/t)(1 - 0{,}0008\theta r/t)(f_y - f_b)t^2$$

In both a) and b),

k $= 0{,}5[1 + e/(n/2 + h)] \leqslant 1$

e is the distance from the centre of the bearing to the end;

n is the bearing length;

h is the web depth;

t is the thickness;

r is the interior bend radius;

θ is the acute angle between the web and the bearing surface;

f_y is the yield strength;

f_b is the stress due to overall bending;

f'_c $= \left(\pi^2 E t^2 / 4h^2\right)[1 - (f_b/f_{bc})^2]$

f_{bc} is the bending stress to cause web buckling (12.1.4.1).

12.5 Curved walls

12.5.1 Tubes in axial compression

The slenderness for local buckling of the wall of a circular tube subjected to longitudinal compressive stress, due to axial force or moment, is given by

$$\lambda = 4(r/t)^{1/2}[1 + (r/t)^{1/2}/35]$$

where

r is the radius;

t is the thickness.

Clause 9.4 shall be used to give the buckling stress. There is no post-buckling strength.

12.5.2 Tubes in radial compression

12.5.2.1 For local buckling of the wall of a circular tube subjected to radial compression, the slenderness is given by

a) for $a/r > 3{,}3(r/t)^{1/2}$

$$\lambda = 6r/t$$

b) for $a/r < 3{,}3(r/t)^{1/2}$

$$\lambda = 3{,}3(a/t)^{1/2}(r/t)^{1/4}$$

where

a is the length of tube;

r is the radius;

t is the thickness.

Clause 9.4 shall be used to give the buckling stress, f_c, which is taken to limit the resistance.

12.5.2.2 Should an allowance for imperfections be required, the characteristic radial pressure, p_k, shall be limited by

$$p_k \frac{r}{t} \left[1 + \frac{1{,}5\delta}{t\left(1 - \dfrac{p_k r/t}{f_e}\right)} \right] = f_y$$

where

δ $= d_{max} - d_{min}$

d is the diameter;

p_k is the design pressure;

f_e $= \pi^2 E/\lambda^2$

λ is the slenderness from 12.5.2.1.

12.5.3 Tubes in shear

For the wall of a circular tube subjected to shear stress, the slenderness for local buckling is given by

a) for $a/r > 9(r/t)^{1/2}$

$$\lambda = 48(r/t)^{3/4}$$

b) for $a/r < 9(r/t)^{1/2}$

$$\lambda = 2{,}8[(a/r)(r/t)^{5/2}]^{1/4}$$

Clause 9.4 shall be used to obtain the buckling stress with $f_o = f_{sy} = 0{,}6 f_y$. There is no post-buckling strength.

13 Torsion

13.1 General

Torsion may be resisted by shear stress (St. Venant torsion) or by the warping rigidity of the member or by a combination of the two modes. Some selected cases of St. Venant torsion are covered in 13.2. For the general problem, reference shall be made to the literature.

13.2 Resistance

Some characteristic resistances, T_k, for the fully plastic state are given below:

a) Round bar

$$T_k = 2r^3(0{,}6 f_y)$$

where r is the outside radius.

b) Rectangular bar

$$T_k = \left(a^2/2\right)(b - a/3)(0{,}6 f_y)$$

where a, b are the dimensions and $a < b$.

c) Round tube

$$T_k = 6r^2t(0{,}6f_y)$$

where

r is the mean radius;

t is the thickness.

d) Closed section

$$T_k = 2At(0{,}6f_y)$$

where

A is the enclosed area;

t is the minimum thickness.

For thin-walled hollow rectangular sections, $0{,}6f_y$ shall be replaced by f_{sc} from 12.4.1.

For thin-walled tubes, $0{,}6f_y$ shall be replaced by f_{sc} from 12.5.3.

14 Bolted and riveted connections

14.1 Use of fasteners

14.1.1 Primary joints in the main structural members shall be made with aluminium, stainless steel, or galvanized or otherwise protected steel bolts, or solid aluminium rivets. Steel bolts shall be used when steel and aluminium are to be joined together, unless the joint is maintained in a dry condition.

14.1.2 Loads shall not be shared between mechanical fasteners and welds in the same connection.

14.1.3 All bolted joints shall be designed to act in bearing when considering the ultimate limit state. The effective bearing area of pins, bolts and rivets shall be the fastener diameter multiplied by the thickness in bearing.

14.1.4 If friction-type joints are required to carry service loads, it shall be demonstrated that the assumed coefficient of friction is available (14.3.6). If slipping of a friction-type joint is to be an ultimate limit state, it shall have the same reliability as a joint in bearing.

14.1.5 Connections and splices should be symmetrical about the axes of the members connected thereby. Members of a framework, meeting at a joint, should be so arranged that their centroidal axes intersect at a point. A group of fasteners connecting two members should have its centre of gravity on the intersection point of the axes of the members. If any of these arrangements is not practicable, provision shall be made for the effect of the resulting eccentricity.

14.1.6 The number of fasteners in line in the direction of the force should not exceed six, with normal spacing, nor the length exceed $15d$ overall.

14.2 Spacing of fasteners

14.2.1 In the rules for spacing rivets and bolts, the nominal diameter, d, of the fastener shall be used; the spacing shall mean the distance centre-to-centre on any gauge line. The centres of fasteners shall be not less than $1{,}25d$ from the edge nor less than $1{,}5d$ from end edges towards which the load is directed. The distance between centres of bolts shall be not less than $2{,}5d$. (See 14.3.3 for the influence of spacing on the resistance.)

14.2.2 If the faying surfaces are to be held in close contact to reduce the possibility of crevice corrosion, the fastener spacing should not exceed $10d$.

14.2.3 For widely spaced fasteners, the compressive resistance controlled by local buckling of the plate between fasteners shall be obtained from 9.4 using the equations in 14.2.3.1 to 14.2.3.4.

14.2.3.1 Where the fasteners are arranged in a rectangular pattern as in figure 9 a), the slenderness is given by the larger of

$$\lambda = 1{,}7s/t$$

$$\lambda = 1{,}3g/t$$

14.2.3.2 Where fastener rows are arranged in a staggered pattern, as shown in figure 9 b), the slenderness for the plate between fasteners is given as follows:

a) for $s/g < 1$

$$\lambda = (1{,}3 + 0{,}6s/g)g/t$$

b) for $g/s < 1$

$$\lambda = (1{,}7 + 0{,}2g/s)s/t$$

14.2.3.3 For fasteners near the edge of a plate, the slenderness of the plate is given by the larger of

$$\lambda = 1{,}6s/t$$

$$\lambda = 3e/t$$

14.2.3.4 In 14.2.3.1 to 14.2.3.3, the symbols are as follows:

g, s are the spacings between the rows of fasteners measured normal to and in the direction of the force, respectively;

t is the metal thickness;

e is the edge distance.

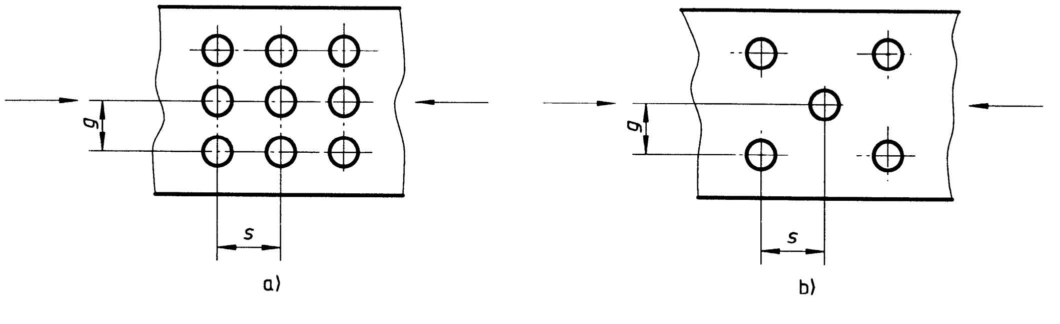


Figure 9 — Measurement of fastener spacing

14.3 Strength of joints

14.3.1 Bolts and rivets in shear

The shear resistance of a bolt or solid rivet is given by

$$R_k = 0{,}6Af_u$$

where

A is the cross-sectional area of the fastener at the shear plane (net or gross, as applicable);

f_u is the ultimate tensile strength of the fastener material.

Special fasteners shall be proportioned to give the same reliability as is provided by this Technical Report.

14.3.2 Bolts and rivets in bearing

The characteristic resistance of a bolt or rivet in bearing is given by the equations in 14.3.2 a) to 14.3.2 d).

a) In the general case:

$$R_k = 2dtf_u$$

b) For load directed towards an edge:

$$R_k = etf_u$$

but not greater than $2dtf_u$

where

e is the end edge distance, $\geqslant 1{,}5d$

d is the hole diameter;

t is the plate thickness;

f_u is the ultimate tensile strength of the connected material.

c) Where the end edge is oblique to the line of action of the force, as in figure 10, the resistance for a single bolt is given by

$$R_k = [e + (e - d)\cos^2\theta]tf_u$$

but not greater than $2dtf_u$

where

e is the normal distance from the hole centre to the end edge;

θ is the angle made by the end edge with the direction of load.

d) In the case of unrestrained single lap joints, the bearing strength is given by

$$R_k = (t + t_1)ef_u/4 \leqslant (t + t_1)df_u/2$$

but not greater than permitted by 14.3.2 a) or 14.3.3 b) for the thinner sheet,

where t, t_1 are the thicknesses of the two sheets.

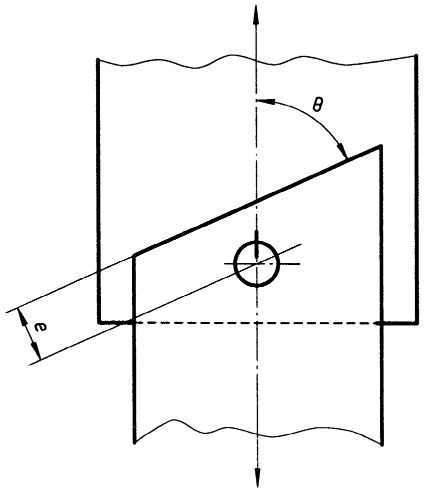


Figure 10 — Distance of single bolt from end edge

14.3.3 Tear-out of bolt and rivet groups ("block shear")

14.3.3.1 For groups of fasteners in a rectangular pattern as shown in figure 11, with load directed towards the edge, the tear-out resistance is given by

$$R_k = [e + (n - 1)(s - d) + (m - 1)(g - d)]tf_u$$

but not greater than $2n'dtf_u$

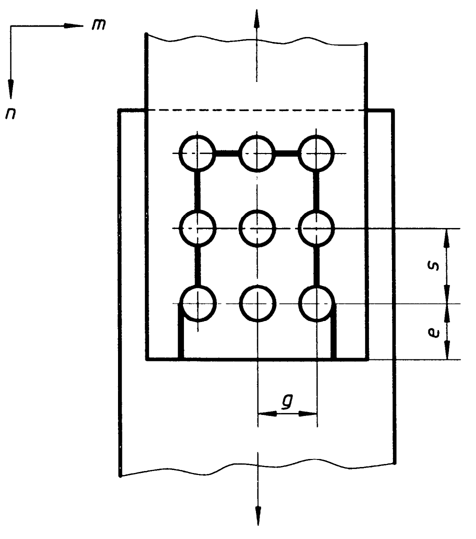


Figure 11 — Tear-out of fastener groups: rectangular pattern

where

e is the edge distance in the direction of stress for the first row (not less than $1,5d$): when $e > 2d$, use $2d$

d is the hole diameter;

s is the fastener spacing measured in the direction of the load;

g is the fastener spacing measured normal to the direction of the load;

m is the number of longitudinal rows of fasteners;

n is the number of transverse rows of fasteners;

f_u is the ultimate strength of the connected material;

n' is the total number of fasteners.

14.3.3.2 For a triangular or trapezoidal group of bolts in a staggered pattern, as shown in figure 12, the tear-out resistance is given by

$$R_k = [2(m-1)\left(g - d + s^2/4g\right) + e]tf_u$$

but not greater than $2n'dtf_u$

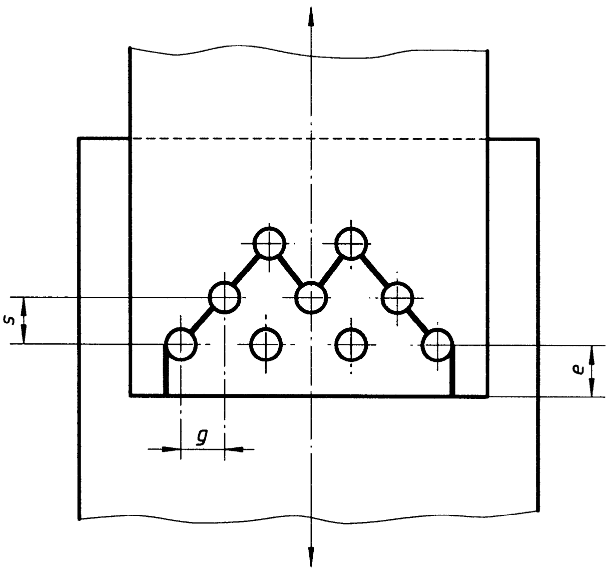


Figure 12 — Tear-out of fastener groups: staggered pattern

where

e is the end distance, or $2d$ if $e > 2d$

m is the number of fasteners in the first transverse row;

g, s are the transverse and longitudinal spacing between rows of fasteners; the spacings between fasteners in the individual rows are $2g$ and $2s$

n' is the total number of fasteners;

d is the hole diameter.

14.3.3.3 For groups of fasteners carrying shear at the ends of webs (figure 13), the resistance of the connected web to "block shear" failure is given by

$$R_k = 0,5(L - nd)tf_u$$

where

L is the length of the shear path;

t is the plate thickness;

n is the number of fasteners in the shear path;

d is the hole diameter;

f_u is the ultimate tensile strength of the plate material.

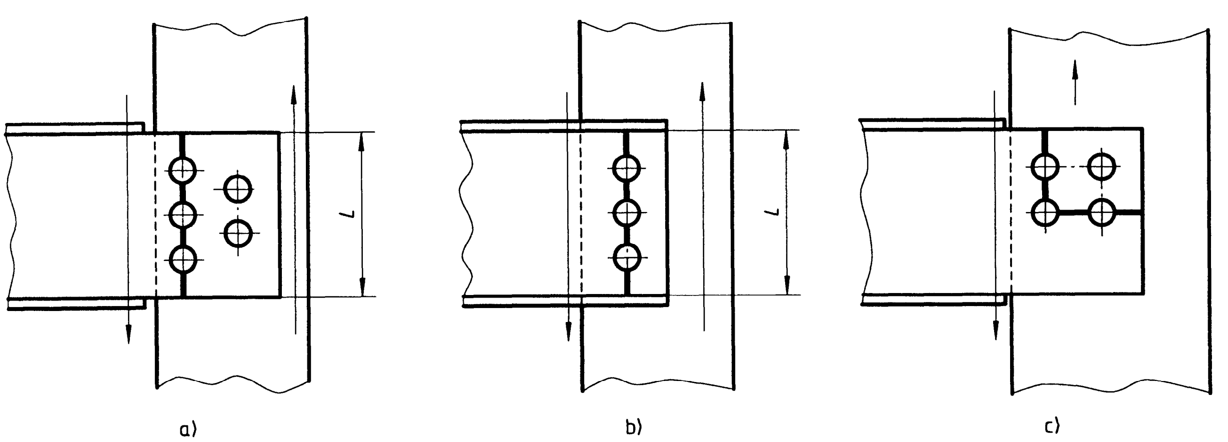


Figure 13 — Block shear load paths

14.3.3.4 In a group of three or more fasteners that lie on a circle, designed to carry a torque in the plane of the fasteners (figure 14), given by

$$T = n'rR$$

the resistance of each bolt, R, when governed by tear-out, is given by

$$R_k = stf_u/2$$

but not greater than $2dtf_u$

where

- s is the centre-to-centre distance between adjacent fasteners on the circle;
- d is the hole diameter;
- t is the plate thickness;
- r is the radius of the fastener circle.

In checking the design using this formula, the load on the fasteners due to the torque shall be computed using only those fasteners that lie on the circle, discounting any fasteners that lie inside the circle.

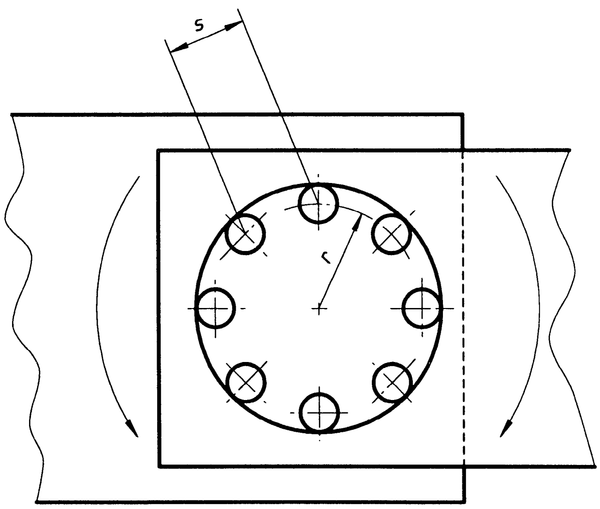


Figure 14 — Fastener group subjected to torque

14.3.4 Concentric force on groups of fasteners

For all groups of fasteners, loaded through the centroid of the group, the applied force shall be assumed to be uniformly distributed between the fasteners.

14.3.5 Eccentric force on groups of fasteners

14.3.5.1 Elastic behaviour

A force, N, applied at a distance, e, from the centroid of a group of fasteners (figure 15), causes the connected part to rotate about a point C lying on the line normal to the direction of the force passing through the group centroid, at a distance, c, from the centroid, on the side away from the force. The value of c is given by

$$c = \Sigma_1^n r_i^2 / ne$$

where

c is the distance from the centroid, O, to the centre of rotation, C;

r_i is the distance of the ith bolt from the centroid;

n is the total number of fasteners;

e is the eccentricity of the applied load.

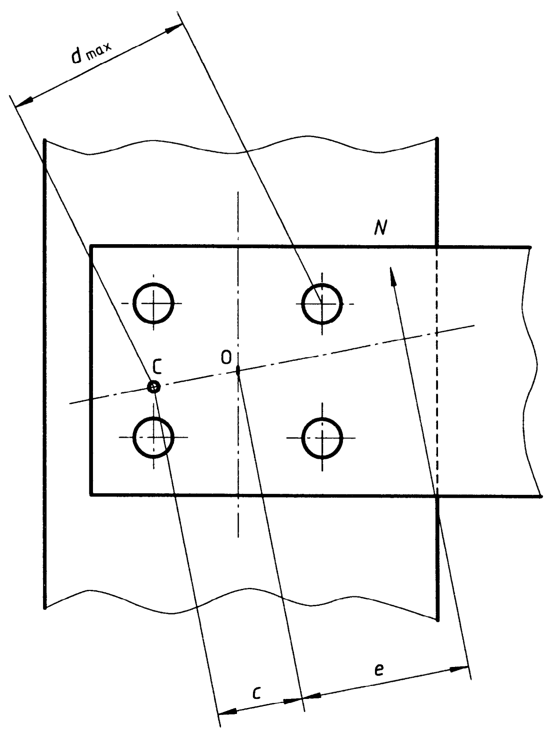


Figure 15 — Eccentrically loaded group of fasteners

a) The highest design force on a fastener, S_d, for elastic behaviour, is given by

$$S_d = (N_{sd}/n)(d_{max}/c)$$

b) For a pure moment, M_{sd}, rotation is about the centroid, and the highest fastener force is given by

$$S_d = M_{sd} d_{max} / \Sigma r_i^2$$

In both a) and b),

d_{max} is the distance from the centre of rotation to the furthest fastener;

N_{sd} is the applied design force;

M_{sd} is the applied design moment.

14.3.5.2 Characteristic static resistance

The same centre of rotation as in elastic behaviour may be assumed at the ultimate resistance, unless the calculated centre lies close to a fastener, in which case the fastener shall be used as the centre of rotation.

The characteristic resistance, N_k, i.e. the value of the applied force at the ultimate limit state, is given by

$$N_k = R_k \Sigma_1^n d_i / (e + c)$$

where

$(e + c)$ is the distance from the centre of rotation, C, to the line of action of the force;

R_k is the characteristic resistance of a single fastener;

d_i is the distance from the centre of rotation, C, to a fastener;

n is the number of fasteners.

14.3.6 Friction-type joints

Where preloaded high-strength bolts are used to create joints that will not slip under service loads, the surface shall be suitably prepared, by means such as sanding, to provide the coefficient of friction adopted.

The serviceability limit load on a bolt in shear, R_s, is given by

$$R_s = 0{,}8 k \mu m A_n f_y$$

where

A_n is the net area of the bolt (stressed area);

f_y is the yield strength of the bolt material;

m is the number of shear planes;

μ is the coefficient of friction;

k = 1 for standard holes,

0,85 for oversize holes,

0,7 for slotted holes.

Installation procedures for preloaded bolts shall ensure that the required bolt tension is realized. The coefficient of friction shall be determined for the surface treatment to be adopted. For preliminary design, a value of 0,3 may be used.

Standard clearances for bolt holes shall satisfy normal manufacturing practice.

Washers are required under the turned element. It is preferable to have washers at both faces in order to reduce local stresses and the possibility of load loss due to creep.

Slotted holes are not to be used to transmit force in the direction of the slot, unless preloaded bolts develop sufficient resistance by friction to provide the required ultimate resistance.

14.4 Fasteners in tension

14.4.1 Rivets will not normally be used in tension.

14.4.2 The resistance of a bolt in tension is given by

$$R_k = A_n f_u \leqslant A_g f_y$$

where

A_n is the net area of the threaded section;

f_u is the ultimate tensile strength of the bolt material;

A_g is the gross area of the bolt cross-section;

f_y is the yield strength of the bolt material.

The net area may be replaced by a "stressed area" in steel bolts, which is somewhat larger than the true net area.

14.4.3 For fasteners in combined shear and tension, the design forces shall be such that

$$N_{sd}/1{,}4R_{dt} + V_{sd}/R_{ds} \leqslant 1{,}0$$

where

N_{sd} is the design tension force;

V_{sd} is the design shear force;

R_{dt} is the design tensile resistance of the bolt;

R_{ds} is the design shear resistance of the bolt.

15 Welded connections

15.1 Alloy selection

The suitability of an aluminium alloy for welding, and the appropriate alloy for the welding wire to be used, shall be established on the basis of

— post-weld strength,

— ductility,

— corrosion resistance,

— control of porosity,

— absence of cracks,

— colour match if anodized.

15.2 Mechanical properties

The strength of a welded joint may be controlled by the heat-affected zone (HAZ) in the base metal, or by the metal of the weld bead.

Table A.1 gives the original mechanical properties of some typical alloys and the properties in the HAZ after welding. In the HAZ, the properties are not related to the welding wire used.

The HAZ shall be assumed to extend 25 mm in each direction from the centre of the weld. This value is to be used for design purposes. If there is evidence that, in a specific case, this requirement is seriously in error, adjustments may be made.

Table A.2 gives the mechanical properties of the weld bead for some typical combinations of base metal and filler alloy.

For any welded joint in tension or shear, the governing resistance shall be the lower of the values for the HAZ of the base metal and for the weld bead.

15.3 Butt joints

Butt joints are usually made with full length, full penetration groove welds, and shall be designed to satisfy 8.2 or clause 9. Partial length and partial penetration groove welds shall be subject to special study.

The reinforcement may be removed to improve fatigue performance but no change in static strength is assumed to occur.

15.4 Fillet welds

15.4.1 Effective dimensions

15.4.1.1 Fillet welds join two adjacent surfaces meeting at an angle lying between 60° and 120°, without edge preparation. If the edges are bevelled, the weld is considered to be of the groove type.

The shortest nominal distance through the weld shall be considered to be the weld throat, a, used to calculate the weld strength.

a) The value of a [figure 16 a)] is given by

$$a = (z - g)/2 \sin (\theta/2)$$

where

z is the fillet weld size;

g is the gap;

θ is the angle between the plates.

b) For a groove weld reinforced by a 45° fillet weld, the throat size [figure 16 b)] is given by

$$a = 0{,}7z(1 + \cot \beta)$$

where β is the groove angle.

15.4.1.2 The effective length of a discontinuous fillet weld, with stops and starts, shall be taken as the true length minus $2z$. The length specified on the drawings shall be the true length.

15.4.1.3 Fillet welds should not be shorter than $8a$. If a shorter weld is unavoidable, the effective throat shall be taken as $L/8$, where L is the true weld length.

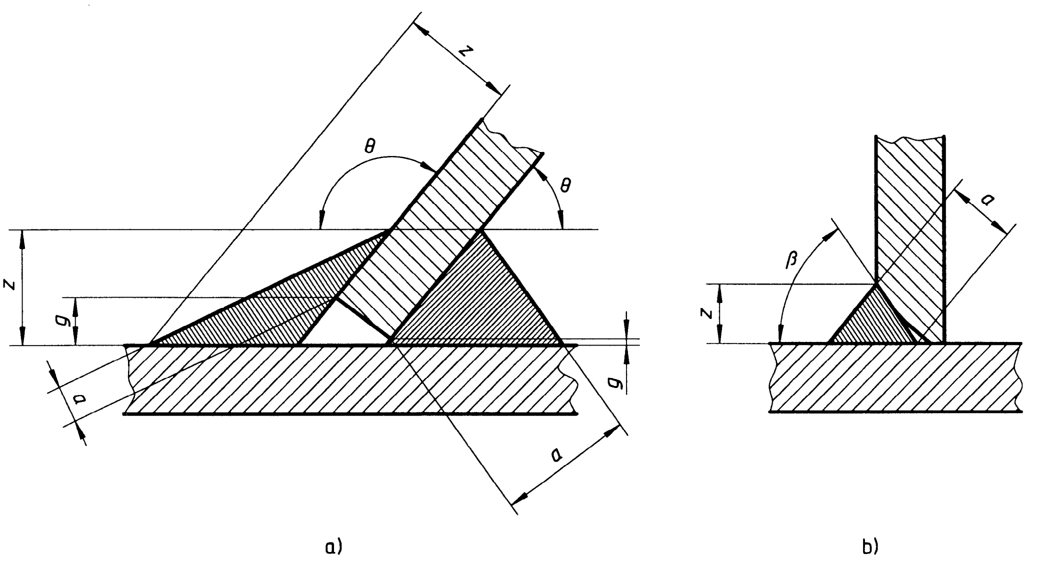


Figure 16 — Throats of fillet welds

15.4.2 Concentrically loaded fillet welds

15.4.2.1 The resistance per unit length of a concentrically loaded fillet weld is given by

$$v_k = kaf_{wu}$$

where

a is the weld throat;

f_{wu} is the ultimate tensile strength of the weld bead;

k is a factor related to the direction of the applied force (figure 17): the values of k may be taken as follows:

= 0,6 for direction X,
= 0,7 for direction Y,
= 0,8 for direction Z.

The values of k are further discussed in annex A.

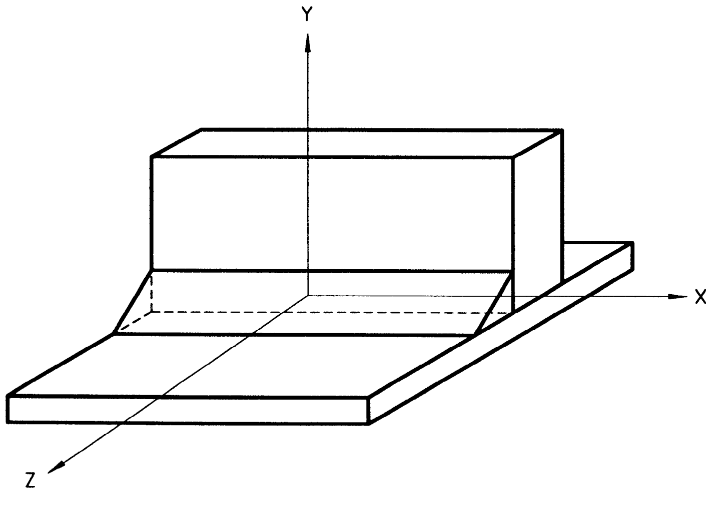


Figure 17 — X, Y, and Z directions of welds

15.4.2.2 For forces applied at an inclination to the weld direction, the components v_{sdx}, v_{sdy}, and v_{sdz} of the design force per unit length shall satisfy the following condition:

$$[(v_{sdx}/0{,}6)^2 + (v_{sdy}/0{,}7)^2 + (v_{sdz}/0{,}8)^2]^{1/2} \leqslant af_{wu}/\gamma_m$$

where γ_m is the partial resistance factor.

Other criteria for oblique forces are discussed in annex A.

15.4.3 Eccentrically loaded fillet welds

15.4.3.1 Moment about Y-axis

15.4.3.1.1 Elastic behaviour

a) For welds subjected to an eccentric load, N, in the X-Z plane (figure 18), applied at a distance e from the centroid of the weld pattern, the connected part rotates about a point, C, at a distance, c, from the centroid, on the line through the centroid normal to the line of action of the force.

The value of c is given by

$$c = I_p/Hae$$

where

I_p is the polar moment of inertia of the weld pattern about its centroid, using a weld width equal to the throat thickness;

H is the total length of the median line of the weld;

a is the weld throat thickness.

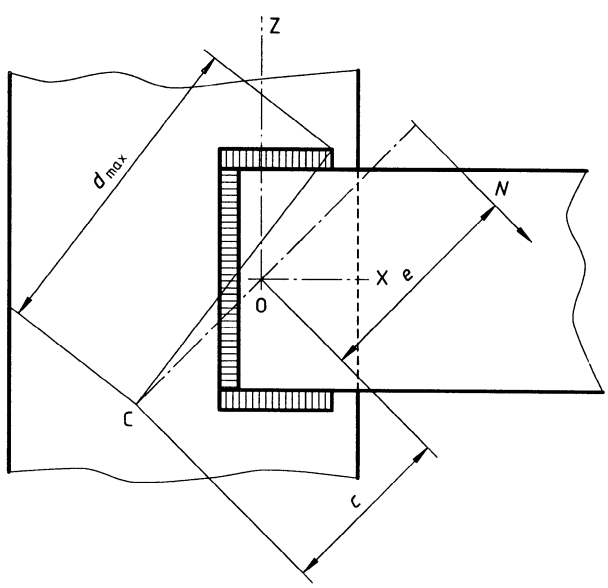


Figure 18 — Welds loaded eccentrically in X-Z plane

The maximum design force per unit length of the weld, v_{sd}, due to an applied design force, N_{sd}, is given by

$$v_{sd} = (N_{sd}/H)(d_{max}/c)$$

b) For pure design moment, M_{sd}, rotation is about the centroid of the weld pattern and the maximum force per unit length of the weld becomes

$$v_{sd} = M_{sd} d_{max}(a/I_p)$$

In both a) and b),

N_{sd} is the applied design force;

M_{sd} is the applied design moment;

d_{max} is the distance from the centre of rotation, C, to the extreme point of the weld.

This value for the force on the weld shall be used when fatigue life is being considered.

15.4.3.1.2 Characteristic static resistance

a) To calculate the characteristic resistance, N_k, the weld is divided into n convenient straight elements on each side of the line passing through the centroid and the centre of rotation, C, obtained using 15.4.3.1. The distances, d_i, from the point C to the mid-points of the elements are determined.

The characteristic resistance is given by

$$N_k = (\Sigma_1^n L_i d_i v_k)/(e + c)$$

where

L_i is the length of the ith element;

d_i is the distance from centre of rotation, C, to the mid-point of the ith element;

v_k is the resistance per unit length of the weld, usually taken as the lowest value from 15.4.2, i.e. for the X-direction of loading;

n is the number of elements.

b) In the case of a pure moment, M_k, the characteristic resistance is given by

$$M_k = \Sigma_1^n L_i d_i v_k$$

where d_i is measured from the centroid of the weld pattern.

15.4.3.2 Moment about Z-axis

For double fillet welds subjected to an eccentric force, N, in the X-Y plane (figure 19), the weld shall be continued around the edges of the plate.

The characteristic resistance is given by

$$N_k = v_{kc} v_{kt} L^2 / 2e(v_{kc} + v_{kt})$$

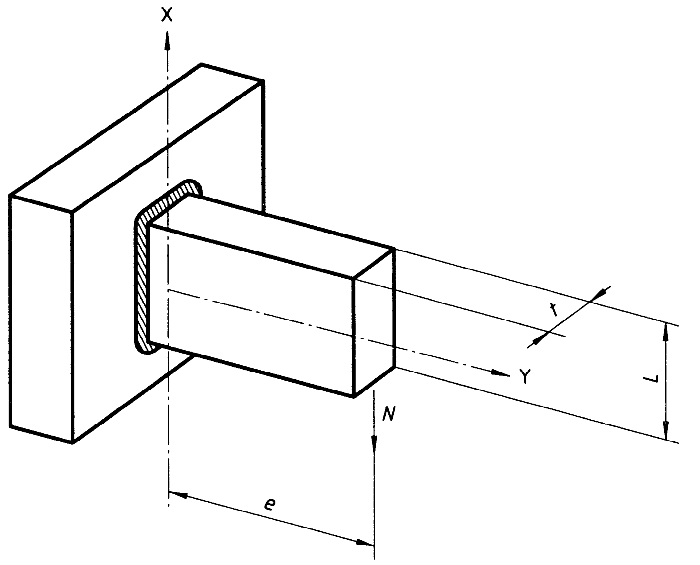


Figure 19 — Welds eccentrically loaded in X-Y plane

where

- e is the eccentricity;
- L is the length of fillet welded joint;
- v_{kc} is the resistance per unit length in compression; this shall be taken as the lesser of the values given by tf_{hu} and tf_y
- v_{kt} is the resistance per unit length in tension at the weld; this shall be taken as the least of the values given by $2a(jf_{wu})$, tf_{hu} and tf_y
- a is the weld throat;
- t is the plate thickness;
- f_y is the yield strength of the base metal;
- f_{hu} is the ultimate tensile strength in the HAZ;
- f_{wu} is the ultimate tensile strength of the weld bead;
- j is a factor related to the combined action of the shear force and the normal force in the weld, given by $j = 0{,}7[1 - (v_x/0{,}6af_{wu})^2]^{1/2}$
- v_x $= N_k/2L$.

In general design, when the influence of the shear force is small, and the fillet weld controls the strength, the resistance may be given by

$$N_k = \left(aL^2/3e\right)f_{wu}$$

15.4.3.3 Moment about X-axis

For double fillet welds bending in the Y-Z plane (figure 20), the characteristic resistance is given by

$$M_k = 0{,}7aL(t + z)f_{wu}$$

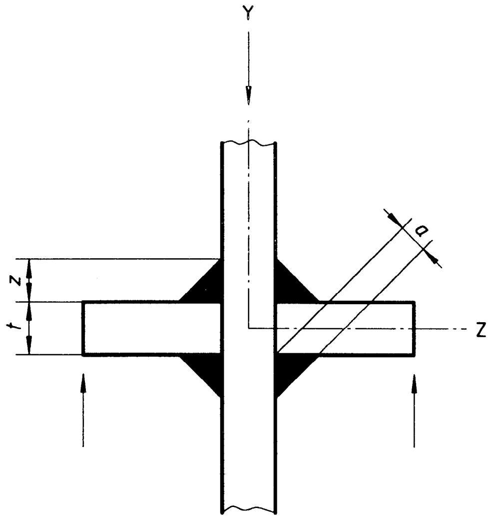


Figure 20 — Welds eccentrically loaded in Y-Z plane

Single fillet welds shall not be subjected to calculated bending forces in the Y-Z plane.

15.5 Flare groove welds

Where welds are to be made between rounded surfaces, as between round bars and at the corners of formed shapes, procedures shall be demonstrated to give the required penetration and throat thickness.

The requirements may be shown to be satisfied by measurement of the weld throat or by load tests.

If measurement is made, the throat shall exceed that required for the design strength by 3 mm.

If tests are made, there shall be at least three specimens made consecutively using the same procedure. The lowest value obtained shall be used as the characteristic strength.

15.6 Slot and plug welds

A connection made by a fillet weld along the inside edge of a hole or slot is acceptable if the radii of the inside corners of rectangular holes are not less than the thickness of the plate plus 5 mm.

The weld shall extend around the full length of the inside edge of the hole.

The length of the weld shall be taken as the length of the centroidal axis of the fillet.

Holes or slots which are completely filled with weld metal shall not be permitted to carry calculated forces.

15.7 Influence of longitudinal welds on overall strength

15.7.1 Tension and compression in constrained members

In concentrically loaded tension members and compression members which do not buckle, longitudinal HAZ zones shall be treated in accordance with 8.2.3.

15.7.2 Beams and columns

For beams, columns and beam/columns, subject to instability, the resistance shall be determined using the gross cross-sectional area in conjunction with a limiting stress given by 9.5.2.

15.8 Influence of transverse welds on overall strength

15.8.1 Where a fillet weld is applied to a plate, and the plate is continuous past the weld as in figure 21, the resistance of the plate shall be the lower of the yield strength of the base metal and the ultimate strength of the HAZ, in both tension and compression.

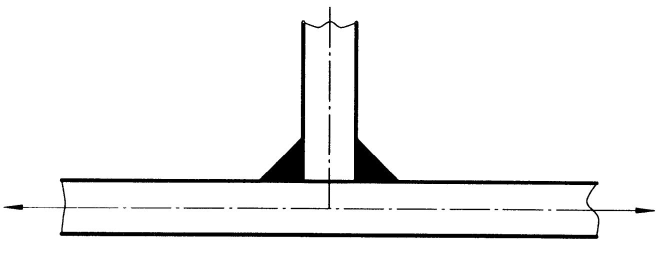


Figure 21 — Weld transverse to stress path

15.8.2 For beams and columns with a transverse weld within $L/5$ of pinned ends or points of contraflexure in the member, where L is the distance between pins or points of contraflexure, the influence of the transverse weld on stability may be disregarded.

For welds near the centre of a column, or near the centre of the compression zone of a beam, the resistance is given by 9.5 for columns and 10.2 for beams, using the yield strength of the HAZ.

15.9 Influence of welds on local buckling

15.9.1 Longitudinal welds may be disregarded where resistance is controlled by local buckling.

15.9.2 In transverse welds without lateral restraint, the applied compressive stress is limited to the yield strength of the HAZ. At restrained transverse welds the compressive stress is limited to the ultimate tensile strength of the HAZ.

Annex A
(informative)

Commentary

NOTE 1 This annex provides commentary on the main body of this Technical Report: clause titles also indicate the relevant clauses in the main body.

A.1 Introduction

There has been a great deal of research activity on the structural behaviour of aluminium throughout the world in recent years, which is finding its way slowly into national standards, but because of the difficulty in obtaining general agreement on the many numerical coefficients and values used throughout a standard, the aim of this Technical Report is to provide means to predict the various modes of failure which have general acceptability, while leaving the choice of values for mechanical properties and coefficients in the design formulae to be selected later, should the Report be used as the basis for an International Standard.

When compared with existing standards, the Report provides a more consistent level of reliability, with simplified, more uniform procedures to determine the ultimate resistance of aluminium components subjected to calculable static forces, based on the concept of limit states design. Design treatments have been selected to give characteristic resistances which have a high probability of being exceeded, the target values being the mean of test results less two standard deviations. Because of the low level of precision of the methods and materials used in structural engineering and the scatter of test results, only two significant figures can be justified for the properties and coefficients in the design formulae.

When using the proposed rules, the designer must include the required partial factors on loads and resistances specified in documents pertinent to the application. As this Report deals only with the strength of aluminium components and connections, without regard to the end use, no information is included on partial factors. Serviceability limit states are not addressed.

A.2 Scope [clause 1]

The Report provides instruction for the design of aluminium components subjected to calculable forces, and is expected to be applied to aluminium assemblies of all types for which there is no separate design code. It is aimed at general engineering areas such as building components, latticed towers, cranes, vehicles, rolling stock and bridges. Aircraft, pressure vessels and other well-established fields have their own bodies of rules.

Only static loads are considered. Dynamic loads and, in particular, design against fatigue failure, will be treated separately.

A.3 Basic design principles [clause 5]

Although the strength of components is discussed in terms of Limit States Design concepts, the expressions provided give the ultimate resistance of aluminium components, and are thus independent of the design philosophy or application. They may be used in the development of rules for working stress design, rated strengths or other approaches for ensuring safe assemblies.

A.4 Basic considerations [clause 6]

A.4.1 Materials [6.2]

A.4.1.1 Aluminium alloys [6.2.1]

The Report deals with wrought alloys in the form of sheet, plate, extrusions and forgings. The design rules proposed may not be applicable to castings, which should be tested when their strength is critical.

A.4.1.2 Fasteners [6.2.2]

There is no restriction on the type of fastener or material used, except that the strength must be known and the material must be compatible with aluminium.

A.4.1.3 Welds [6.2.3]

Filler alloys for welds are restricted to those permitted in the prevailing standards for welded aluminium construction.

A.4.2 Properties [6.4]

A.4.2.1 Physical properties [6.4.1]

The elastic modulus, E, varies by less than 5 % over the range of alloys listed, and a uniform value of 70 000 MPa is adopted. Poisson's ratio, ν, for aluminium is approximately 1/3. The shear modulus, G, derived from these values, is 26 000 MPa, to the same accuracy as the value used for the elastic modulus.

The coefficient of thermal expansion, α, varies between 0,000 023 for the 6xxx series of alloys and 0,000 024 for the 7xxx series. The higher value is used.

Density, ρ, varies between the alloys treated by $\pm$ 2 % from 2 700 kg/m^3.

A.4.2.2 Mechanical properties [6.4.2]

A.4.2.2.1 Specified properties [6.4.2.1]

Mechanical properties to be used in engineering calculations will normally be those claimed for the alloy and temper in the specifications for the registered composition. Alloy designations may be registered with any of a number of national and international bodies, and the same alloy may be given different designations by the different authorities. It is not thought to be necessary to include all the variations of all the alloys in current use: only a limited number of the more popular alloys is included in table A.1, identified by their AA designations. For other alloys, reference should be made to the technical literature provided by the industry.

For a given alloy and temper, there is often a variation in the mechanical properties between different products and thicknesses, and between different directions of loading. In table A.1 the values have been chosen to represent those for typical products and sizes used in general engineering.

The mechanical properties of aluminium alloys that are specified in industry standards are the ultimate tensile strength, f_u, the tensile yield strength, f_y, the elongation in tension, δ, and, in some cases, the bend factor.

The properties of the base metal are established at values for which 99 % of the material is expected to conform at a 0,95 confidence level. This is the A-basis defined in [53].

"Elongation" and "bend factor" are not used in this Report, but the values for the alloys listed are such that ductile behaviour can be anticipated and plastic design methods may be appropriate [26].

Table A.1 — Tensile properties of base metal and HAZ

Stresses in MPa

Alloy	Temper		Base metal		HAZ	
			Yield f_y	Ult. f_u	Yield f_{hy}	Ult. f_{hu}
Sheet and plate						
3003	H112		65	115	35	100
	H14	H1D	115	140	35	100
	H18	HH	165	180	35	100
3004	H112		65	160	60	150
	H32		145	190	60	150
	H34		170	220	60	150
	H36		190	240	60	150
5052	H32		160	215	70	170
	H34	H3D	180	235	70	170
5083	H112	M	125	275	125	270
	H321		215	305	125	270
	H343	H3D	270	345	125	270
5086	H112		125	250	105	240
	H32		195	275	105	240
	H34		235	300	105	240
5454		M	100	215	90	220
	H112		125	220	90	220
	H32		180	250	90	220
	H34		200	270	90	220
5754	H24		200	270	80	190
Extrusions						
6005A	T5		215	260	105	170
6060		TF	150	200	80	140
6061	T6	TF	240	260	120	195
6063	T5	TE	110	150	70	120
	T6	TF	170	205	70	120
6082		TF	260	310	110	210
6106	T5		200	240	105	180
6351	T6	TF	255	290	110	180
7004	T1		205	325	165	280
	T51		255	290	165	280
7020		TE, TF	290	350	210	280
Cast alloys						
AlSi 12		M	70	160	70	160
AlSi 7 Mg		TF	200	230	110	175
AlCu 4 MgTi		TB	250	290		

Table A.2 — Ultimate tensile strength of weld beads, f_{wu}

Stresses in MPa

Filler alloy	Base metal alloy												
	3003	3004	5052	5083	5086	5454	6060	6061	6063	6082	6351	7004	7020
4043	100						150	170	120	190	170	210	210
5356	100	150	170	240	230	220	160	190	120	210	190	260	260
5183				260									

A.4.2.2.2 Derived properties [6.4.2.2]

In some publications the full range of measured values for the tensile, compressive and shear strengths are provided, and use can be made of these values when they are guaranteed. Properties other than the tensile strengths, however, are not usually guaranteed as they are not measured in production. For this reason properties derived from the tensile values are preferred.

A.4.2.2.2.1 Yield strength in compression [6.4.2.2.1]

Compressive yield strength is taken to be equal to that in tension, although, as a consequence of the Bauschinger effect in rolled products of work-hardened alloys, the compressive yield strength may be as much as 10 % below that in tension. Similarly, to facilitate design, no account is taken of the variation between tensile and compressive properties for stress along and across the machine direction of rolled plate.

A.4.2.2.2.2 Yield and ultimate strengths in shear [6.4.2.2.2]

Shear yield strength, f_{vy}, is related to the guaranteed tensile yield strength by the von Mises criterion, using $1/\sqrt{3}$ rounded to 0,6, which is reasonably consistent with the measured values. Theoretically there is no justification to use von Mises for the relationship between the utlimate strengths, particularly as the tensile ultimate is an artificial stress, the true stress being $f_u/(1 + \delta)$, where δ is the elongation at the peak load. However, the factor of 0,6 applied to the ultimate tensile strength gives values that vary between − 10 % and + 5 % of the measured values for the ultimate shear strength, f_{vu} [53] which is acceptable. This is further discussed under A.13.3.1.

A.4.2.2.3 Ultimate strength in bearing on fasteners [6.4.2.2.3]

Bearing strength, f_b, is based on the ultimate tensile strength [31], [44] as it is governed by shear failure in the connected material, up to an end edge distance in excess of two diameters. Beyond this, for design purposes, the value is assumed to remain constant, although it is known to be conservative. No use is made of a "yield strength" in bearing, as distortions are small and are of no concern at the ultimate limit state. In those studies which attempt to relate bearing strength to distortion, the values for the work-hardened alloys become extremely conservative. Comparisons with the tests reported [63] show reasonable agreement for the tests conducted on heat-treated alloys, although the interpretation differs. Should a bolted splice be used at the centre of a column, it is possible that the "softness" of a joint governed by bearing may influence the capacity of the strut; however, the larger partial factor usual for joints makes this unlikely. The fastener itself is not subject to bearing failure, as the fastener can only fail in shear, no matter how thin the connected part or how weak the fastener alloy.

A.4.2.3 Welded properties [6.4.2.3]

Mechanical properties for welds are not provided with the alloy specification and must be obtained from other sources. For engineered assemblies, welding is carried out using manual or automatic, TIG, MIG or plasma arc methods. Because of the different levels of heat-input between these methods, there is some variation in the strength of the resulting joints. To accommodate all these variations within a design standard is impractical because at the time the design is being prepared the method of welding may not be known, and may differ between manufacturers of the same product. It is thus reasonable to specify a single value, for each alloy and combination of alloys, which can be expected to be realized by a qualified welder, regardless of the method used. If higher values can be assured, they may be adopted.

Yield strength of welds may be measured using a 50 mm gauge length, or, as is used in the USA, a 250 mm gauge length. It is evident that the second method gives a higher value. Because yielding in the HAZ is of interest when determining effective sections, and where it may lead to instability, it is important that the value should represent as closely as possible the behaviour of that zone: thus the first method is preferred.

Design values for butt welds may be based on the measured strength of welds or obtained by an arbitrary factor applied to the base metal properties. In this Report an effort has been made to provide values for the strengths that can be expected to be achieved in practice, based on the evidence available. In tables A.1 and A.2, the proposed values for the yield strength, f_{hy}, and ultimate strength, f_{hu}, for the heat-affected zone, and the ultimate strength of the weld bead, f_{wu}, are taken from a qualifying code for welders, CSA-W47.2-M87, or are compromises between the strengths obtained from other documents [9], [27], [37], [44], [48], [55], [70]. In special cases higher values may be justified but will be subject to confirmation.

A similar argument is used for the extent of the heat-affected zone. This can vary widely between automatic MIG welds and manual TIG welds, but without an exact knowledge of the procedures to be used, the designer must assume a value which is reasonable for welds in general. A band, 25 mm wide [37] each side of the weld is now widely adopted because it is simple, typical, and any change from this value has little influence on the final design. Should welding procedures in a particular case give a consistently wider or narrower HAZ, the value may be revised.

A.5 Methods of analysis [clause 7]

Elastic analysis is always permitted even though the force/deformation relationships for the components, up to failure, may not be linear. The justification is that so long as equilibrium is satisfied, and there are no "brittle" components, the solution will be a lower bound. Caution may be needed in redundant lattice structures [64].

As a generalization, aluminium assemblies do not possess the range of ductility enjoyed by structural steel. This is due to the lower spread between yield and ultimate strength, the lower elongation at rupture, the reduction in strength at welds and the significance of the ratio of net area/gross area. The reliance placed on the redistribution of stress after yielding in steel components cannot always be transferred to aluminium and a clear grasp of the limits that can be exploited is required if use is to be made of plastic or ultimate strength methods.

Because many components are not subject to direct analysis and design, proof by testing is acceptable in all cases.

A.6 Characteristic resistance in tension [clause 8]

A.6.1 Bolted construction [8.1]

A.6.1.1 Concentric force [8.1.1]

With the move to limit states design, it is increasingly recognized that the elastic limit does not necessarily limit the load-carrying capacity. If only a small portion is in the non-linear range, overall distortions are small and "uncontrolled deformations" do not occur. For this reason the strength of tension members is governed by two distinct conditions:

— overall yielding of the gross section,

— rupture at the connection.

The "yield" strength is usually that of the base metal, but may be the weighted average yield strength for sections with longitudinal welds (8.2.3).

Rupture may occur across the net section of a bolted joint at the ultimate strength of the base metal, or through a transverse weld at the ultimate strength of the weld.

A.6.1.2 Net sections [8.1.2]

To establish the net section of plates in tension, across a row of staggered holes, the holes are deducted and $s^2/4g$ added for each space. This was shown [14] to represent an upper bound solution based on plastic behaviour.

A.6.1.3 Eccentric force [8.1.3]

A.6.1.3.1 General case [8.1.3.1]

In the general case, the limiting conditions are first yield in the gross section using elastic analysis and the attainment of the ultimate strength on an effective net section based on elastic analysis.

A.6.1.3.2 Special cases [8.1.3.2]

A.6.1.3.2.1 Eccentrically loaded gusset plates [8.1.3.2.1]

It is assumed that the plate is capable of becoming fully plastic, up to yielding of the gross section and up to the ultimate tensile strength at the net section.

A.6.1.3.2.2 Single angles [8.1.3.2.2]

The strength of single angles loaded through one leg is from [43], using plastic analysis up to the ultimate tensile strength at the net section. In this reference it is also demonstrated that a lug angle attached to the outstanding leg, in an attempt to harness the strength of that leg, is ineffective.

A.6.1.3.2.3 Single channels, T-sections and double angles connected eccentrically [8.1.3.2.3]

A channel section connected by the web, in the fully plastic condition, if the ends are pinned, can be shown to have an effective area equal to the web area plus 0,4 times the flange areas. Allowing for some end fixity permits the effective net area to include half the flange areas.

A.6.1.4 Combined tension and bending [8.1.4]

Compact sections which become fully plastic are treated by the relationships due to [38].

A.6.2 Full penetration butt welds [8.2]

A.6.2.1 Full width transverse welds [8.2.1]

A butt weld is composed of the weld bead and the associated base metal. These zones may have different strengths, making it necessary to specify values for both the weld bead and the HAZ. Because the weld bead is narrow and yielding results in little overall distortion, the ultimate tensile and shear strengths are of primary interest. It follows that the strength of the joint is given by the lower of the yield strength of the base metal and the ultimate strength of the weld, which is the lower of the weld bead strength and the HAZ strength, from tables A.1 and A.2. This follows [27].

Although partial penetration butt welds are not recommended in engineered aluminium assemblies, should the need arise they may be treated in the same manner as groove welds and the weld throat may be taken to be the groove depth, or the throat created must be demonstrated in trial welds.

A.6.2.2 Full width oblique welds [8.2.2]

Because tension in ductile materials is controlled by shear at 45°, oblique welds up to an inclination of 45° have the same strength as an orthogonal transverse weld. Beyond this, the strength increases and can be calculated on the basis of the weighted average yield strength across a right section. However, the shear stress along the weld may still limit the resistance if the weld strength is much below the yield strength for the base metal. To give a consistent relationship, the component of shear stress along the weld is limited to $f_{wu}/2$.

A.6.2.3 Longitudinal welds [8.2.3] [9.5.2] [10.1.1 a)]

As only part of the cross-section is affected by longitudinal welds, the reduction in yield strength at the weld may be accounted for in two ways:

a) The mean stress is limited to the weighted average yield strength of the cross-section [37] given by

$$f_o = f_y - (A_h/A_g)(f_y - f_{hy})$$

b) The thickness of the metal in the heat-affected zone is reduced in the ratio f_{hy}/f_y to give an effective cross-sectional area:

$$A_n = A_g - A_h(f_y - f_{hy})/f_y$$

In both a) and b),

A_g, A_h are the gross area and the area of the HAZ;

f_y, f_{hy} are the yield strengths of the base metal and the HAZ, respectively.

Both methods are used in the Report, the first for column design and the second for tension and bending.

A.7 Characteristic resistance in compression [clause 9]

A.7.1 Concentric force [9.1]

The mean axial stress is governed by the buckling stress, f_c, which is a function of a limiting stress and the theoretical elastic buckling stress.

A.7.2 Limiting stress [9.2]

Where there is no instability, the resistance is controlled by one of several limiting stresses, f_o. These stresses are related to yielding and local buckling, except in fully restrained transverse welds where the ultimate strength of the HAZ provides the limit.

A.7.3 Normalized slenderness [9.3]

All modes of buckling are related to a normalized slenderness, $\bar{\lambda}$, which is a function of the member proportions, the stress distribution, the boundary conditions and the limiting stress.

A.7.4 Normalized buckling stress [9.4]

There are two limiting conditions which provide boundaries for buckling stress under the action of compressive force:

a) a limiting strength, f_o, usually the yield strength, f_y;

b) the theoretical elastic buckling stress, f_e, given by

$$f_e = \pi^2 E/\lambda^2$$

This elastic buckling stress is normalized with respect to f_o, leading to the normalized slenderness, $\bar{\lambda}$, given by

$$\bar{\lambda} = (f_o/f_e)^{1/2} = (\lambda/\pi)(f_o/E)^{1/2}$$

where

E is the elastic modulus;

λ is a geometric ratio called the slenderness: for simple struts

$$\lambda = L/i,$$

in which

L is the length,

i is the radius of gyration.

Between these two boundaries of yielding and elastic buckling lies the actual curve for the buckling stress for a particular column profile and alloy. A widely accepted model for the relationship is that known as Perry-Robertson, used in the UK for many years, most recently in [12]. In its original form this method says simply that the limiting axial force, N, occurs when the stress on the concave side of a strut, with an initial bow of δ, reaches the limiting stress, f_o. The condition is given by

$$\frac{N}{A} + \frac{N\delta}{S(1 - N/N_e)} = f_o$$

where N_e is the theoretical elastic buckling load.

This can be expressed as

$$f_c + \eta f_c/(1 - f_c/f_e) = f_o$$

where

f_c is the mean axial stress at failure;

η $= \delta c/i^2$

c is the distance from the centroid to the extreme fibre in compression;

f_e is the elastic buckling stress.

Normalizing the relationship with respect to f_o and extracting f_c/f_o gives the normalized buckling stress, $\bar{f}$:

$$\bar{f} = f_c/f_o = \beta - \left(\beta^2 - 1/\bar{\lambda}^2\right)^{1/2}$$

where

$$\beta = \left(1 + \eta + \bar{\lambda}^2\right)/2\bar{\lambda}^2$$

Replacing η by $\alpha(\bar{\lambda} - \bar{\lambda}_o)$ gives

$$\beta = \left(1 + \alpha(\bar{\lambda} - \bar{\lambda}_o) + \bar{\lambda}^2\right)/2\bar{\lambda}^2$$

This modified form of η permits the creation of a range for $\bar{\lambda} < \bar{\lambda}_o$ in which buckling can be disregarded. Variations in the value of α permit the fitting of the curve to the actual behaviour of struts as determined by direct tests or computer simulation.

[51] and [52] have provided a wealth of information on the buckling of aluminium struts, drawn from many sources, which has been used as the basis for [27] for aluminium structures. Results from column tests are given in the form of normalized stress, $\bar{f}$, plotted against the normalized slenderness, $\bar{\lambda}$. At each value of the slenderness, the mean test value is given, with the values at two standard deviations each side of the mean. There are several interpretations of the test results that may be used to develop a design curve. The two that have been considered are a best fit to the mean less two standard deviations, and a lower bound to all test results. The important test results are mostly for I-shapes failing about the weak axis, which are known to give lower buckling stresses than those for flexure about the strong axis: thus the curve will give reasonably conservative values relative to the entire family of strut shapes.

Using the Perry-Robertson formula, regression analyses have been conducted to establish the most suitable values of the parameters, α and $\bar{\lambda}_o$. There are many variations which might justify different buckling curves but the most significant is the difference in behaviour between heat-treated and work-hardened alloys, and, historically, two curves have been provided. This has been treated [52] where it is argued that the curves should be lower bounds to the test results. This leads to the values of the coefficients in column 1 of table A.3.

To facilitate design in general, it is convenient to have a range of slenderness within which buckling need not be considered. One choice is to make $\bar{\lambda}_o = 0{,}3$. Holding this value constant, the values of α in column 2 in table A.3 give the best fit curves to the mean of the test values less two standard deviations.

Table A.3 — Coefficients in buckling formulae

Alloy		Column 1 Lower bound	Column 2 $\lambda_o = 0{,}3$
Heat-treated alloys:	α	0,2	0,2
	$\bar{\lambda}_o$	0,1	0,3
Work-hardened alloys:	α	0,32	0,4
	$\bar{\lambda}_o$	0	0,3

Figures A.1 and A.2 show these proposed curves, with the test values. Included in the figures are the straight line formulae used in [3] which can be closely represented by

— heat-treated $\bar{f} = 1{,}12 - 0{,}37\bar{\lambda}$

— work-hardened $\bar{f} = 1{,}15 - 0{,}48\bar{\lambda}$

It is seen that the proposed design stresses are well below those used in North America, particularly in the zone for values of $\bar{\lambda}$ between 1 and 2. The straight line tangent modulus formula, while taking full account of the non-linear material properties, takes too little account of imperfections. It does not necessarily lead to unsafe designs but there is a variation in the reliability index over the range of applicability of the formula.

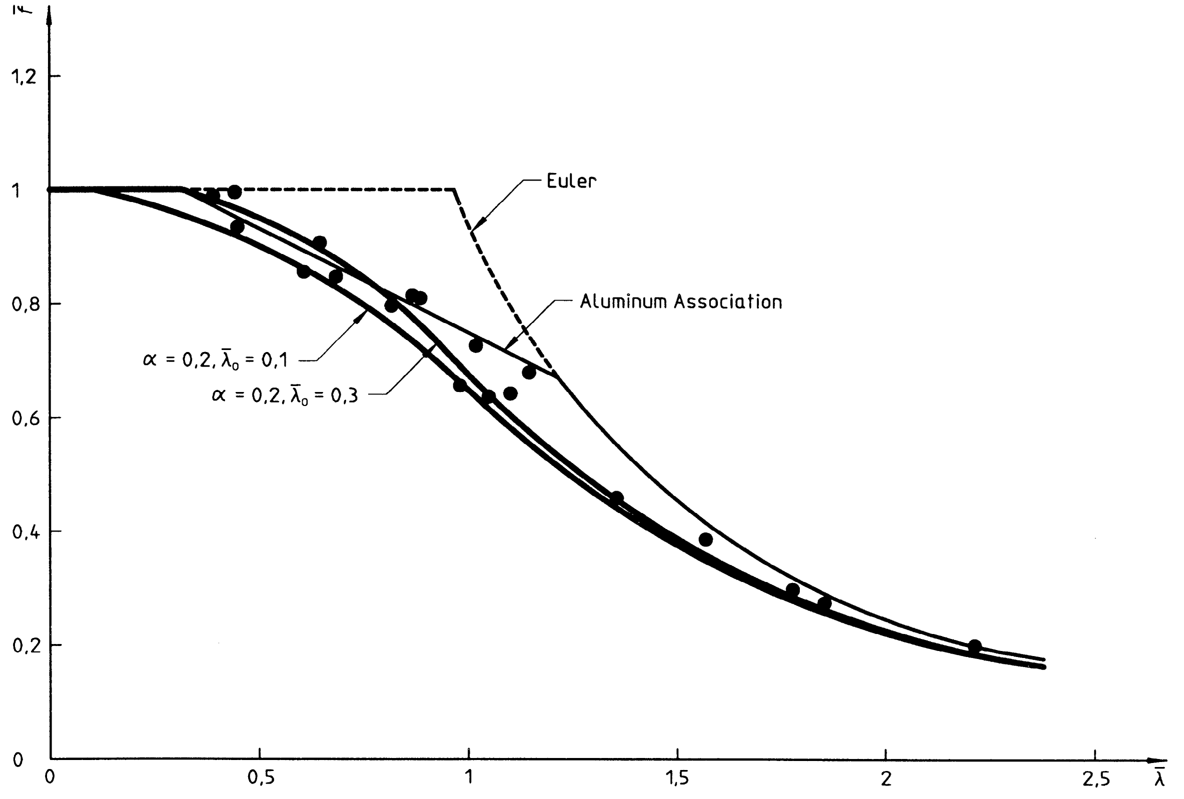


Figure A.1 — Column buckling curves with test results: heat-treated alloys [51]

In other standards, for steel and aluminium, different buckling curves or reduction factors are occasionally used for such variations as profile shape, direction of buckling, symmetrical and asymmetrical shapes, and mode of buckling. In this Report it is proposed that only the two main alloy groups justify separate buckling curves for columns, and that flexural and torsional modes of buckling are equally well served by the same curves.

That there is an effect attributable to the shape is well established, but the reasons for the effect are not so clear. One possible explanation is that in the development of the Perry-Robertson formula there is a factor, $\eta = \delta c/i^2$, that initially represented geometric properties of the column and its imperfection, but was later used to represent the material properties, becoming $\alpha\bar{\lambda} = (\alpha/\pi)(f_0/E)^{1/2}(L/i)$ in the normalized form. The ratio of the extreme fibre distance, c, to the radius of gyration, i, will evidently have an effect on the capacity.

Computer simulation performs what the Perry-Robertson formula models, with the added input of non-linear material behaviour; however, an arbitrary initial imperfection of $L/1\,000$ for all slenderness ratios will automatically give a higher curve for strong axis bending. In the Perry-Robertson formula the imperfection is a function of the slenderness.

Another consequence of normalizing the buckling curves is that the yield stress is now included in the parameter, η, causing the value of η to increase with the yield strength, thereby lowering the buckling curve, working contrary to the accepted view that a higher yield stress usually means a sharper knee in the stress/strain diagram which will raise the buckling curve.

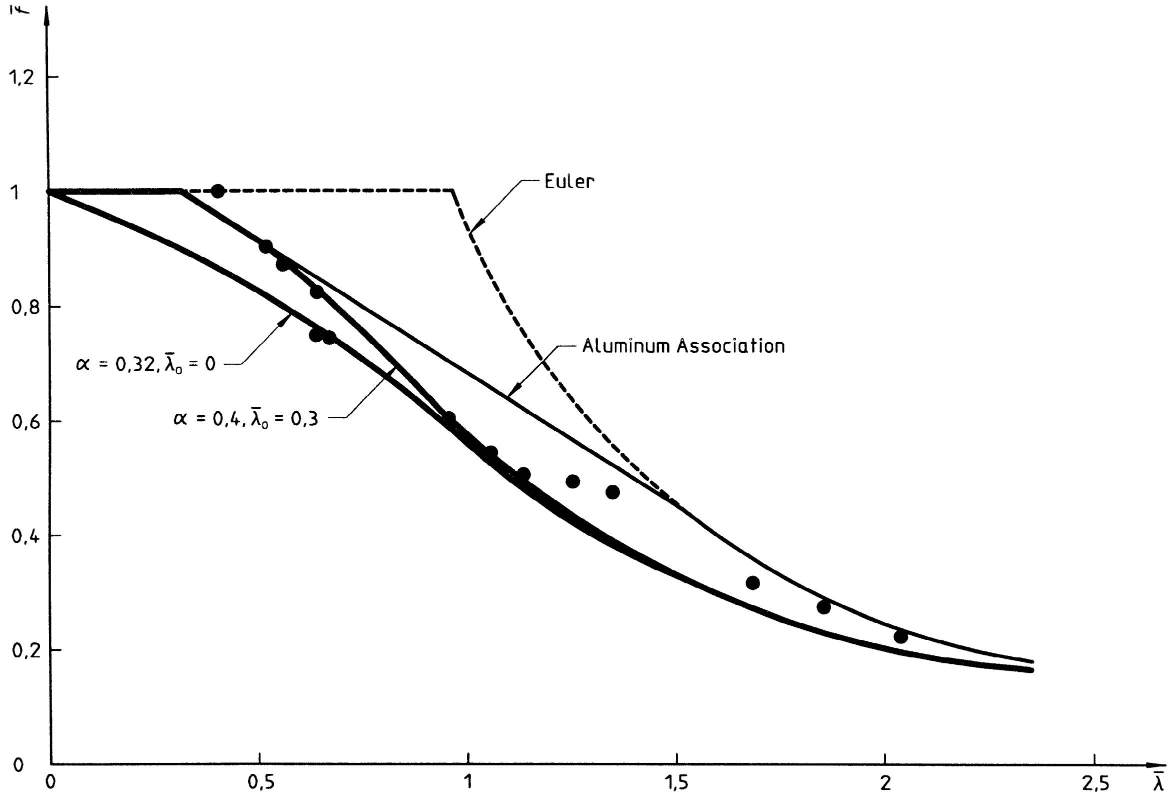


Figure A.2 — Column buckling curves with test results: work-hardened alloys [51]

In [27] it is suggested that the ratio between the stresses for buckling in unsymmetrical and symmetrical sections may be expressed as

$$K_1 = 1 - \rho\psi^2 \frac{\bar{\lambda}^2}{\left(1 + \bar{\lambda}^2\right)(1 + \bar{\lambda})^2}$$

where

$\psi = (c_1 - c_2)/h$

c_1 and c_2 are the distances to the extreme fibres;

h is the section depth;

ρ varies with the alloy type.

This reduction can exceed 10 %. No equivalent ratio has been proposed for a relationship between weak and strong axis flexure, as has been advanced for steel columns.

Extruded aluminium shapes are available in such a wide variety that it may not be practical to recognize them all. As the buckling curve is based on the results for weak axis bending, and already represents the lower levels of column behaviour, there may be no call for any further reductions to the basic curve, other than that due to welding. [27] gives characteristic resistances for struts that can be as much as 30 % below those currently used in North America.

A.7.5 Flexural buckling of columns [9.5]

A.7.5.1 General case [9.5.1]

To establish the axial force to cause the flexural buckling of a column it is necessary to know the slenderness, λ, and the limiting stress, f_o, to obtain the normalized slenderness, $\bar{\lambda}$, and hence the normalized buckling stress, $\bar{f}$. The characteristic axial resistance is then:

$$N_k = \bar{f} f_o A_g$$

In most cases the limiting stress, f_o, will be the yield strength of the base metal, but other influences may arise, in particular welding and local buckling.

A.7.5.2 Influence of longitudinal welds [9.5.2]

The influence of longitudinal welds takes two forms. In both heat-treated and work-hardened alloys, the heat of welding reduces the yield strength and creates residual stresses.

With longitudinal welds, the weighted average yield stress, discussed under 8.2.3 and 9.2, is used as f_o in the buckling formula. This strength reduction represents the primary influence of longitudinal welds on the capacity of a column. The additional effect, due to the creation of longitudinal residual stresses, was studied in [51] giving curves, obtained using computer simulation, of the influence of residual stresses on a fully annealed alloy, in which there is no reduction in yield strength due to welds. This influence can reasonably be applied as an added factor for all alloy types. Treating the family of curves developed as a statistical set, the mean reduction in strength and coefficient of variation at $\bar{\lambda} = 1$ are 0,076 and 0,03. At $\bar{\lambda} = 2$ there is a negligible mean influence.

A proposed reduction factor is

$$k = 0{,}9 + 0{,}1 \left| 1 - \bar{\lambda} \right| \leqslant 1{,}0$$

which is approximately mean less one standard deviation. In view of the high values used for residual stresses, and the superposition of this factor on the influence of the reduced yield strength, the resulting characteristic strength is considered to be sufficiently conservative. It is to be observed that although the weld reduces the yield strength, the residual stress at the weld is tension, creating compressive stress elsewhere; thus welds at the extreme fibre may not be the most detrimental. For this reason, weld location is not taken into account in determining the effective strength in struts.

An alternative procedure in [51] is simply to apply a reduction factor, K_2, to the value given by the buckling curve for the base metal.

For heat-treated alloys:

$$\bar{\lambda} \leqslant 1, \quad K_2 = A_r/A - (A_r/A - 0{,}85)\bar{\lambda}$$

$$1 \leqslant \bar{\lambda} \leqslant 3, \quad K_2 = 0{,}775 + 0{,}075\bar{\lambda}$$

where

A_r is the effective area $= A - (1 - f_{hy}/f_y)A_h$

For nonheat-treated alloys (for all tempers):

$$\bar{\lambda} \leqslant 1, \quad K_2 = 1 - 0{,}2\bar{\lambda}$$

$$1 \leqslant \bar{\lambda} \leqslant 3, \quad K_2 = 0{,}7 + 0{,}1\bar{\lambda}$$

A.7.5.3 Influence of local buckling that leads to failure [9.5.3]

Local buckling in an outstanding flange limits the extreme fibre stress to the value that initiates buckling, f_c. This stress will be used as f_o in the overall column buckling formula [1]. How local buckling stress is determined is discussed in A.10.1 (clause 12.1).

A.7.5.4 Influence of local buckling with post-buckling strength [9.5.4]

Post-buckling strength is only considered in flat elements that have both longitudinal edges supported. The limiting stress is the mean stress when the stress at the boundaries of the element reaches the yield strength. The derivation is given in A.10.2.2 (clause 12.2.2).

A.7.6 Torsional and torsional-flexural buckling of columns [9.6]

The expressions for elastic torsional or torsional-flexural buckling of columns, used to obtain the slenderness, are taken directly from [72]. Non-linear material behaviour and imperfections are assumed to have the same influence as they have for flexural buckling, permitting the use of the same normalized buckling formulae. This is known to be conservative.

A.7.6.1 Open sections symmetrical about one axis [9.6.1]

For sections symmetrical about one axis only, such as channel shapes, buckling in flexure about the axis of symmetry combines with torsion. The interaction equation has been chosen to give a reasonable match to the correct relationship from [72]:

$$\lambda^4 - \left(\lambda_x^2 + \lambda_t^2\right)\lambda^2 + \left[1 - \left(\frac{x_o}{t_o}\right)^2\right]\lambda_x^2\lambda_t^2 = 0$$

A.7.6.2 Angles, T-sections and cruciform sections [9.6.2]

Torsional buckling in sections which possess little warping rigidity is related closely to the slenderness ratio $\lambda = 5b/t$, where b is the longest leg. For extruded shapes, the value of b may be measured from the beginning of the root radius, because of the benefit of the increase in torsion constant attributed to the root. For angles formed from strip, the value of b is the full leg width measured to the median line of the adjacent leg, and the slight negative effect due to the bend radius is neglected.

A.7.6.3 Influence of warping resistance [9.6.3]

The contribution made by warping rigidity to the stability of open sections with one axis of symmetry is taken from [72]. It is expected that the designer will have tables of formulae giving torsion and warping constants.

A.7.6.4 Asymmetrical open sections [9.6.4]

Although the flexural-torsional buckling of asymmetrical open sections has an established theory [72], it is not felt that these seldom-used sections merit the space that the theory demands. When the need arises, the theoretical elastic buckling stress can be calculated and the normalized slenderness established.

A.7.7 Built-up compression members [9.7]

Shear flexibility in built-up compression members such as battened or stitch-bolted multiple bar members, double channels and double angles, is accounted for in the manner indicated in [72] and [13].

A.7.8 Lattice columns [9.8]

Lattice columns are treated in the same manner as other columns and beam-columns, using the buckling stress of a chord as the limiting stress. Shear flexibility of the latticed planes is neglected as its influence is small.

A.8 Bending [clause 10]

A.8.1 Moment resistance [10.1]

The resisting moment of a section may be limited by a fully plastic condition, as in compact sections, or by local buckling, as in thin-walled sections. Although the design procedures themselves will establish what the limiting stress will be, for a given set of proportions, it is convenient to classify the geometries that determine the types of behaviour under compressive stress.

In practice, the most common limit is the yield strength at the extreme fibre; the behaviour is still basically elastic with no significant permanent set. This is the design condition when local buckling will not occur prior to yielding. The buckling formula for plates requires that the normalized slenderness be less than 0,5. Straining beyond the yield strength is not permitted because the drop in elastic modulus at yield may precipitate local buckling.

Where there are longitudinal welds, the effective section (8.2.3) is used to give the effective section modulus. If the extreme fibre stress does not exceed the yield strength in the base metal, the effective thickness of the HAZ could vary with its distance from the neutral axis. For simplicity, all the HAZ's are reduced in thickness to $(f_{hy}/f_y)t$, as is required for the fully plastic moment. It is suggested that the centroid of the gross section be used, as the resulting error is too small to justify carrying out the tedious task of determining the true position of the neutral axis at each stage of yielding.

Should the section be sufficiently compact, yielding need not lead to instability and large strains in the plastic range are possible, permitting the development of the fully plastic moment. The requirement for this stability is that the normalized slenderness for local buckling of the element be less than 0,3. For outstanding flanges a typical slenderness for local buckling is provided by $3{,}5(w/t)$, giving a normalized slenderness of $1{,}1(w/t)f_y/E)^{1/2}$. If this is equal to 0,3, then $w/t = 0{,}27(E/f_y)^{1/2}$, which compares with the recommendation in [39] that w/t be limited to $0{,}3(E/f_y)^{1/2}$.

When the elements have a normalized slenderness greater than 0,5, local buckling must be considered in the parts carrying compressive stress. This applies also to lattice masts in which chord buckling controls the strength.

Bending resistance may be controlled by the compressive or the tensile stress, thus for each class there are two limiting conditions to be considered. The fully plastic moment of the gross cross-section can only be realized if the stress at the net section does not exceed the ultimate tensile strength. The two conditions are independent limits to the resisting moment. In computing the net properties, the holes with fasteners in the compression zone are not deducted, and the neutral axis of the gross section is used.

Buckling of an outstanding flange usually precipitates overall buckling and thus limits the capacity. For box beams and other shapes with compression elements supported on two long edges, the effective thickness is used (12.2.2) to give the effective section modulus.

A.8.2 Lateral-torsional buckling [10.2]

A.8.2.1 [**10.2.1**] Beams that have a relatively low lateral strength may fail by lateral-torsional buckling when bent about the strong axis if torsional restraint or lateral restraint to the compression flange is not provided.

a) For beams subjected to a uniform moment, free to buckle laterally, the slenderness is derived using the basic elastic formula from [72]. This slenderness is normalized with respect to the limiting stress in the beam, and the value obtained is used in the formula for the buckling of columns, for the appropriate alloy type. Figure A.3 shows how the values predicted using this procedure compare with the test results of [22].

An alternative approach is to take the normalized slenderness in the form:

$$\bar{\lambda} = (M_o/M_e)^{1/2}$$

where

M_o is the moment to develop the limiting stress;

M_e is the theoretical elastic buckling moment.

[33] showed that a curve given by the formula:

$$\overline{M} = 1/\left(1 + \bar{\lambda}^4\right)^{1/2}$$

where $\overline{M} = M_c/M_o$, is an equally good fit to the test results of [22].

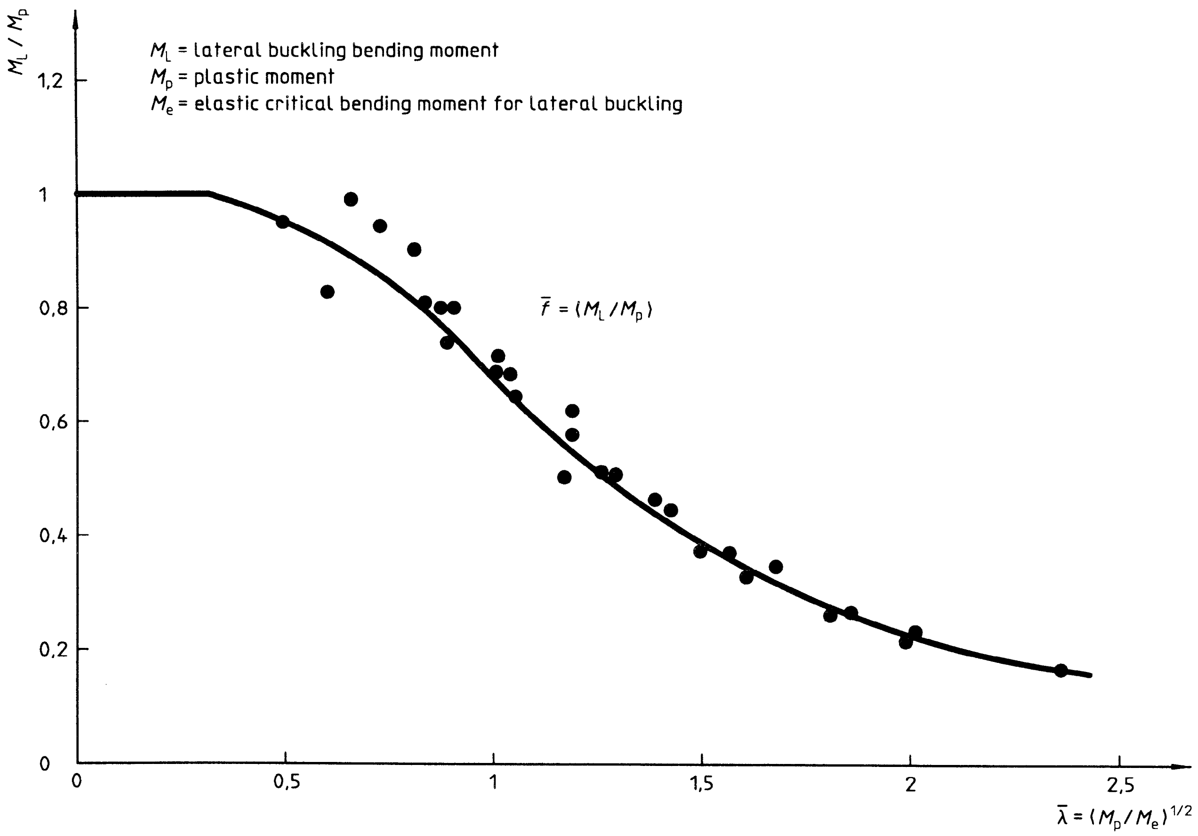


Figure A.3 — Lateral-torsional buckling curves with test results: 2014-T6 I-beams [22]

The general formula for the slenderness to give the elastic critical stress under a uniform moment is

$$\lambda = \frac{(W_x L)^{1/2}}{[I_y(0{,}04 I_t + C_w/L^2)]^{1/4}}$$

For I-sections it is possible to simplify this expression by using the following approximations:

$$I_t \approx b_t^3 \quad i_x = h/2 \quad C_w = I_y h^2/4 \quad I_y = A i_y^2 = b^3 t/6 \quad W_x = Ah/2$$

where A is the area of two flanges.

By extracting the flexural component, the slenderness can be expressed as

$$\lambda = \frac{L/i_y}{[1 + (Lt/bh)^2]^{1/4}}$$

For deep rectangular, hollow or solid, beams a simple expression is obtained using the torsion constant $I_t \approx 4I_y$ with the warping rigidity $C_w \approx 0$:

$$\lambda = 2{,}2(i_x/i_y)(L/h)^{1/2}$$

b) If only the tension flange is restrained, a beam may buckle laterally by rotating about the restrained flange. The method in [72] is used to give the critical elastic moment, M_e, neglecting any rotational constraint.

In this type of buckling, the contributions to the resistance made by the lateral flexural stiffness and by the torsional stiffness are independent. The critical stress is the sum of the individual values attributable to the two sources of rigidity: thus both values must be zero for the critical stress to be zero. The expression that gives the critical elastic moment can be broken into the components of flexural and torsional resistance, and the slenderness, λ, will satisfy the relationship:

$$\frac{1}{\lambda^2} = \frac{1}{\lambda_1^2} + \frac{1}{\lambda_2^2}$$

where

λ_1 is the slenderness of the flange for lateral flexure = L/i_y

i_y is the radius of gyration about the weak axis;

λ_2 is the slenderness for torsional buckling of the beam.

For buckling by twisting about the tension flange, resisted by St. Venant torsion only, the critical stress is given by

$$F_e = \frac{G I_t}{I_p}$$

For I-sections the torsion constant, $I_t \approx bt^3$, the polar moment of inertia, $I_p \approx bth^2$, and the shear modulus, $G = E/2(1 + 0{,}33)$. Equating the critical stress to $\pi^2 E/\lambda_2^2$ gives

$$\lambda_2 \approx 5h/t$$

Using $i_y \approx 0{,}3b$ leads to the simplified expression for the slenderness for lateral buckling:

$$\lambda = (L/i_y)/\left\{1 + 0{,}5[(Lt)/(bh)]^2\right\}^{1/2}$$

A.8.2.2 [**10.2.2**] For a linear moment gradient, caused by moments applied at the ends, where the maximum applied moment occurs at the end rather than near the mid-point of the member, the value of the equivalent uniform moment to be compared with the critical moment follows steel practice and is taken to be [76]:

$$0{,}6M_{max} + 0{,}4M_{min}$$

Should the beam be bent in double curvature, M_{min} is negative. In no case is the equivalent moment taken as less than $0{,}4M_{max}$.

For other variations from the basic case, such as the location and distribution of the applied forces and moments, end conditions and elastic restraints, reference should be made to the literature to obtain the critical elastic moment, M_e. The normalized slenderness is then obtained directly from $\bar{\lambda} = (M_o/M_e)^{1/2}$ to obtain $\bar{f}$ and hence the actual buckling moment, $M_c = \bar{f}M_o$.

A.9 Beam-columns [clause 11]

When making comparisons between the different methods used to limit the values of forces in combination, it is important that the predictions for the combined loads be used, not those for the individual components. Should a pair of forces N and M be such that the limiting condition is satisfied by a design procedure, and the values to satisfy an alternative procedure are $(1 + \beta)N$ and $(1 + \beta)M$, i.e. both forces receive the same factor, then β represents the discrepancy between the procedures. Between the competing proposals addressed in preparing this Report, the value of β rarely exceeded 0,05 in the critical ranges.

A.9.1 Moment and axial force [11.1]

A.9.1.1 [**11.1.1**] Under the action of a compressive axial force and a lateral force, the maximum stress in a member with simple end supports is given by

$$N/A + M/W(1 - N/N_e) = f_{max} \leqslant f_o$$

a) For failure in the plane of bending, if the stresses are elastic up to the limiting stress, f_o, then the above condition applies. However, for compact members, limited by yielding, the behaviour is not linear elastic up to the yield strength, and the combination of axial force, N, and applied moment, M, is controlled instead by the formula [34]:

$$N/N_{dx} + M/M_d(1 - N/N_{ex}) = 1$$

where

N_{dx} is the axial resistance for buckling in the plane of the applied bending moment;

N_{ex} is the elastic value for this resistance;

M_d is the moment to cause yielding or is the fully plastic moment, depending on the proportions of the section.

This relationship was extensively studied [51] and was shown to provide a conservative design procedure over the entire range. This treatment is applicable to compact shapes in which local buckling is not a factor.

b) When the behaviour is elastic up to attaining the limiting stress in the extreme fibre, as in lattice beam-columns and thin-wall sections, the original formula is applicable.

c) When the stress in the extreme tension fibre reaches the yield strength first, the formula is recast to reflect this behaviour.

A.9.1.2 [11.1.2] In the case of beam-columns failing by lateral-torsional buckling, the mode of failure is distinct from that of in-plane failure, and the interaction formula becomes [35], [51]:

$$N/N_{dy} + M/M_{dl}(1 - N/N_{ex}) = 1$$

where

N_{dy} is the axial resistance for weak axis buckling;

M_{dl} is the moment resistance controlled by lateral buckling.

A.9.1.3 [11.1.3] For axially loaded members with moments about both principal axes, use is made of the beam-column formula for failure in the plane of bending, with the addition of a term for the second moment. Failure by combined flexure and torsion is not treated and will require reference to the literature [72], to determine the elastic buckling force needed to calculate the normalized slenderness.

A.9.2 Eccentrically loaded columns [11.2]

A.9.2.1 General case [11.2.1]

Columns with eccentric axial loading are treated as beam-columns. For eccentricity in two directions which leads to failure in combined torsion and flexure, reference may be made to [72].

A.9.2.2 Single angle struts [11.2.2]

For the case of discontinuous single angle struts loaded through one leg, the failure mode is that of lateral-torsional buckling [43].

A.9.3 Shear force in beam-columns and eccentric columns [11.3]

Shear force due to lateral loads is increased by the action of the axial force in the same manner that the bending moment is increased, and the same factor is applicable [72].

Shear force in eccentrically loaded columns is derived from the geometry of the deflected column.

A.10 Local buckling [clause 12]

A.10.1 Flat elements in compression [12.1]

A.10.1.1 Slenderness [12.1.1]

Flat elements will theoretically buckle elastically when the compressive stress reaches a value [72]:

$$f_e = \frac{k\pi^2 D}{b^2 t}$$

where

k is a factor;

D $= Et^3/12\left(1 - v^2\right)$

b and t are the breadth and thickness;

E is the elastic modulus;

v is Poisson's ratio.

Equating this stress to an expression of the Euler type,

$$\frac{k\pi^2 D}{b^2 t} = \frac{\pi^2 E}{\lambda^2}$$

gives:

$$\lambda = \pi \left(\frac{12\left(1 - \nu^2\right)}{k} \right)^{1/2} \frac{b}{t} = m \frac{b}{t}$$

This form of the slenderness, λ, is used for all local buckling in flat elements. In this case the limiting stress is always the yield strength, f_y, leading to a normalized slenderness:

$$\bar{\lambda} = (f_y/f_e)^{1/2} = (\lambda/\pi)\,(f_y/E)^{1/2} = (b/t)\,(m/\pi)\,(f_y/E)^{1/2}$$

Using this value, the normalized buckling stress, $\bar{f}$, is obtained from the appropriate normalized buckling curve for plates. Such a curve is provided by the same Perry-Robertson formula as that used for columns but with the parameters modified. The values suggested are:

Heat-treated alloys

$\alpha = 0{,}2 \quad \bar{\lambda}_o = 0{,}5$

Work-hardened alloys

$\alpha = 0{,}4 \quad \bar{\lambda}_o = 0{,}5$

Curves illustrating these formulae to give the initial buckling stress are shown in figure A.4.

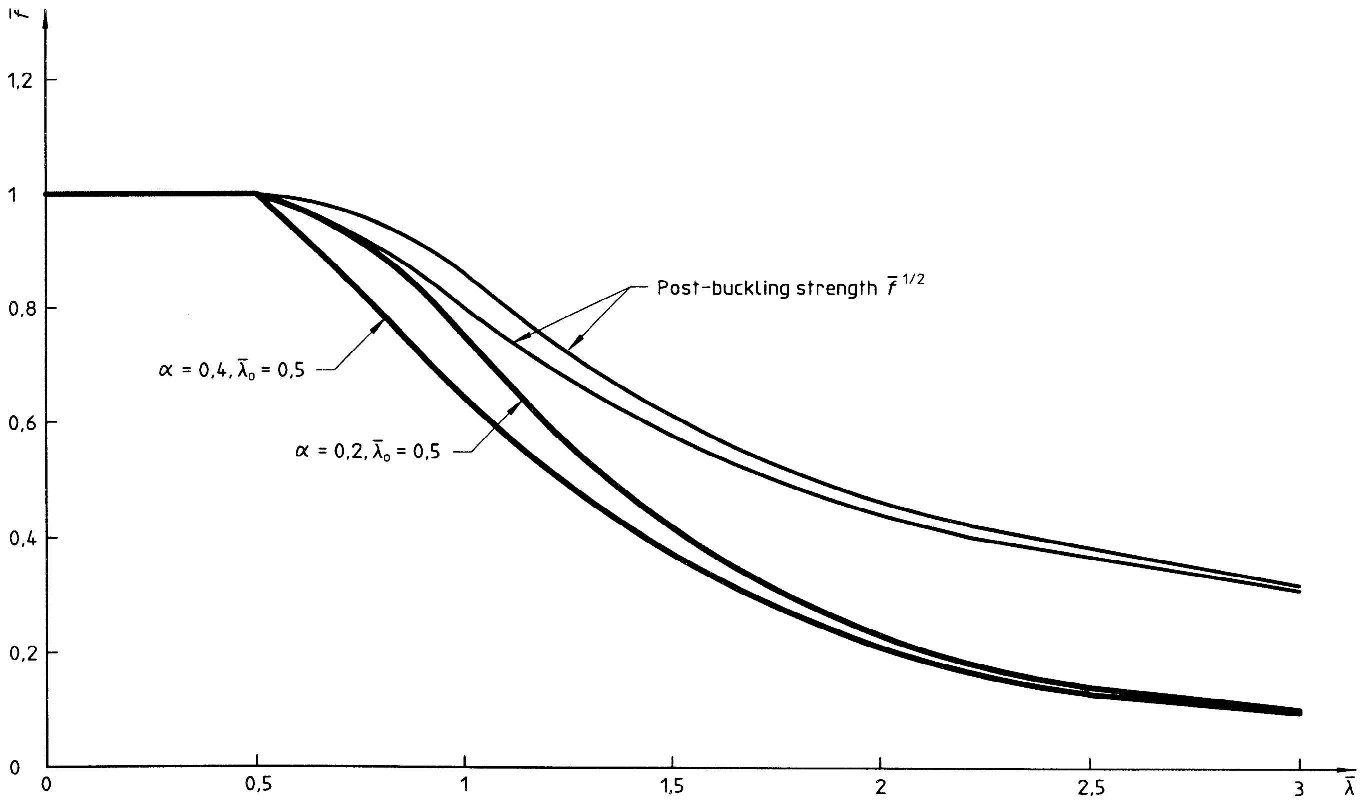


Figure A.4 — Initial buckling and post-buckling curves for plates

A.10.1.2 Uniform axial compression [12.1.2]

A.10.1.2.1 [12.1.2.1]

For a flat element with both long edges simply supported, the value of m is 1,63. Where there is a direct connection to adjacent elements, the restraint increases the critical stress. For an element of width b, subjected to uniform stress, in sections of uniform thickness, the theoretical values of m in [71], are compared in table A.4 with those given by the expression in the Report.

A.10.1.2.2 [12.1.2.2]

For an element free at one long edge and simply supported at the other, the value of m is 5. When a flange is attached to a web the constraint increases the critical stress. The expression $m = 3 + 0{,}6(a/b)$ has been chosen to match the values given in [71] with reasonable accuracy in the practical range. In table A.5, the theoretical values of m are compared with those used in the Report for outstanding flanges in sections of uniform thickness, subjected to uniform compressive stress.

Local buckling in outstanding flanges usually leads to collapse, and this is assumed to hold in the general case. It follows that where flange buckling occurs in columns or beams that are subject to overall flexural buckling, this initial local buckling stress, f_c, will be used as the limiting stress, f_o, in the column formula.

A.10.1.3 Uniform compression due to bending [12.1.3]

For the flanges of sections in bending, the web provides greater restraint than when the stress is uniform throughout the section and the coefficients are adjusted to reflect this, based on [71].

Table A.4 — Theoretical v. design values of m for plates

a/b		2	1	0
Columns	m		1,63	1,24
	$1{,}2 + 0{,}4a/b$		1,6	1,2
Beams	m	1,63		1,24
	$1{,}2 + 0{,}2a/b$	1,6	1,4	1,2

Table A.5 — Theoretical v. design values of m for outstanding flanges

a/b	0	1	2	3
m	2,9	3,5	4,0	4,8
$3 + 0{,}6(a/b)$	3,0	3,6	4,2	4,8

A.10.1.4 Non-uniform stress distribution [12.1.4]

A.10.1.4.1 Both long edges simply supported

In the case of a linear stress gradient, as caused by in-plane bending of elements simply supported at both long edges, that varies from f_1 to f_2 across the element, where f_1 is the maximum compressive stress, the theoretical values for m derived from [72] are compared in table A.6 with those given by the simplified expressions used in the Report.

A.10.1.4.2 One long edge simply supported, other edge free

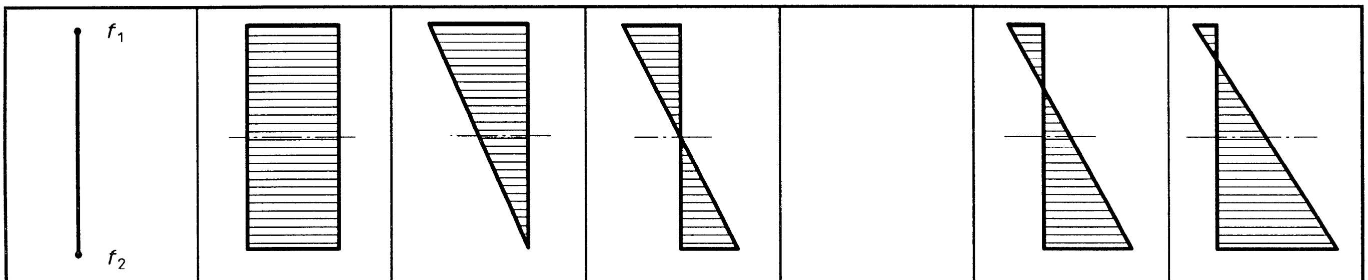
For elements with only one long edge simply supported, when the stress varies linearly across the element, failure is by torsional buckling. (Local buckling, as such, does not occur in practical elements with one edge free and the other simply supported i.e. angle, tee, and cruciform shapes.) The values of m given in the Report are derived using the methods in [72]. How m varies with the stress distribution is illustrated in table A.7.

It is seen that in the two extreme cases there is no buckling. These values ignore any restraint at the supported edge and thus apply strictly only to equal angles bending about the minimum axis.

A.10.1.4.3 Flanges

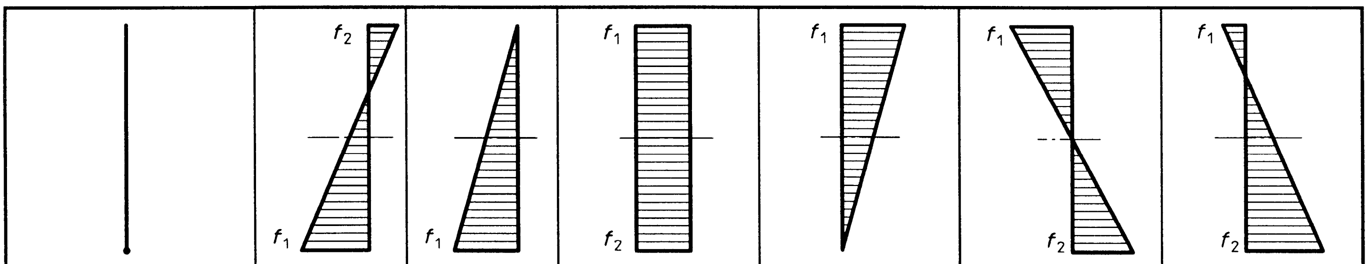
The expression presented is for the case of a stress that varies from zero at the web of a section to a maximum compression at the extreme fibre, as in an I-section bending about its weak axis.

Table A.6 — Theoretical v. design values of *m*, both long edges supported



f_2/f_1	1	0	− 1	f_2/f_1	− 2	− 4
m $(1,1 + f_2/2f_1)$	1,63 1,6	1,17 1,1	0,67 0,6	m $1,2/(1 - f_2/f_1)$	0,44 0,4	0,27 0,24

Table A.7 — Values of *m* for one long edge supported



f_2/f_1	− 1/3	0	1	0	− 1	− 3
m	0	2,5	5	4,3	3,5	0
	$2,5(1 + 3f_2/f_1)^{1/2}$			$2,5(3 + f_2/f_1)^{1/2}$		

A.10.1.5 Sandwich panel skins [12.1.5]

The skin of a sandwich panel, bonded to a flexible core such as one composed of foamed plastic, behaves as a plate on an elastic foundation, for which the critical stress is given theoretically by

$$f_e = 0{,}86(EE_cG_c)^{1/3}$$

where E_c and G_c are the elastic properties of the core material.

Because this mode of buckling is very sensitive to imperfections, the factor is reduced to 0,5 [61]. Equating the modified expression to the Euler formula gives the slenderness:

$$\lambda = 4{,}5E^{1/3}/(E_cG_c)^{1/6}$$

This is used with the yield strength for the alloy to obtain the buckling stress from the curves for plate buckling.

A.10.2 Post-buckling strength of flat elements in compression [12.2]

A.10.2.1 Outstanding flanges [12.2.1]

Local buckling in outstanding flanges usually precipitates overall collapse, unless the member is fully constrained against such a failure.

A.10.2.2 Flat elements supported on both long edges [12.2.2]

The development of post-buckling strength occurs primarily in flat elements supported at both long edges. In such elements, after initial buckling the stress distribution no longer remains uniform, although the transverse effective axial strain distribution probably does. As the force increases, the stress towards the boundaries rises, and, in the limit, reaches the yield strength. For design purposes the magnitude of the total force can be represented by either

a) the product of the gross area and an effective stress; or

b) by the product of the yield strength and an effective area.

The device is artificial but useful. [74] suggested that the limiting force is given by

$$R = bt(f_e f_y)^{1/2} = f_y b\left[\left(\frac{f_e}{f_y}\right)^{1/2} t\right]$$

Predictions using this concept are unconservative in the elasto-plastic range, and to bring them into line with test results the elastic buckling stress, f_e, is replaced by the actual buckling stress, f_c. The relationship can then be interpreted in two ways, by an effective strength or by an effective area, as follows.

a) An effective strength given by

$$f_m = (f_y f_c)^{1/2} = \bar{f}^{1/2} f_y$$

This stress is used as f_o, in conjunction with the gross area, to establish the overall stability of columns and beams.

b) An effective thickness given by

$$t' = (f_c/f_y)^{1/2} t = \bar{f}^{1/2} t$$

This thickness is used when calculating the effective section modulus which, when multiplied by the yield strength, gives the ultimate moment in beams where there is no overall instability.

In these expressions, $\bar{f}$ is the normalized local buckling stress, considered in 12.1.

The relationship obtained using this method is compared with that used for steel in figure A.5, shown with some test results [41], and is compared with test results for aluminium in figure A.6 [54].

How the post-buckling strength compares with the initial buckling stress is shown in figure A.4.

A.10.3 Elements with stiffeners [12.3]

A.10.3.1 Lipped flanges [12.3.1]

The treatment for lipped flanges, derived from [65] and [49], models the flange as an element rotating about the intersection of the flange and web, with elastic restraint provided by the web. The moment of inertia of the lip is taken about the inner face of the flange element as this gives values that agree closely with the warping constant required.

A common case is that of simple 45° lips for which the expression for the slenderness is modified slightly to become:

$$\lambda = \frac{5b}{t}\left(\frac{1+3\beta}{1+\beta+3{,}7\left(\dfrac{0{,}5\beta^3(b/t)^2+0{,}1}{\alpha+0{,}5}\right)^{1/2}}\right)^{1/2}$$

It is to be observed that bend radii have little or no influence on the critical stress and are disregarded in this treatment.

A.10.3.2 Stiffened panels [12.3.2]

In stiffened panels subjected to compression, it is usual for the stiffeners to run in the direction of loading, as transverse stiffeners are much less effective. Treating the panel as an orthotropic plate [72] and neglecting the torsional stiffness [40], [66] leads to a buckling force per unit width given by

$$N_{cr} = 2\frac{\pi^2}{b^2}(D_1D_2)^{1/2}$$

where D_1 and D_2 are the longitudinal and transverse rigidities respectively.

This gives the elastic buckling stress and is used to derive the expressions for the slenderness.

A.10.4 Flat elements in shear [12.4]

A.10.4.1 Buckling stress [12.4.1]

Initial elastic buckling of a rectangular panel, $a \times b$, $a > b$, with simply supported boundaries, subjected to uniform shear stress, f_{se}, occurs at a stress [72]:

$$f_{se} = 5{,}35[1+0{,}75(b/a)^2]\pi^2Et^2/12\left(1-v^2\right)b^2 = \pi^2E/\lambda^2$$

Thus:

$$\lambda = 1{,}4(b/t)/[1+0{,}75(b/a)^2]^{1/2}$$

The normalized slenderness is then $\bar{\lambda} = (f_{sy}/f_{se})^{1/2}$, where f_{sy} is the shear yield strength, $0{,}6f_y$. This normalized slenderness is used with the buckling curve for plates to obtain the normalized buckling stress, $\bar{f}$, and hence the actual initial buckling stress:

$$f_{sc} = \bar{f}f_{sy} = \bar{f}(0{,}6f_y)$$

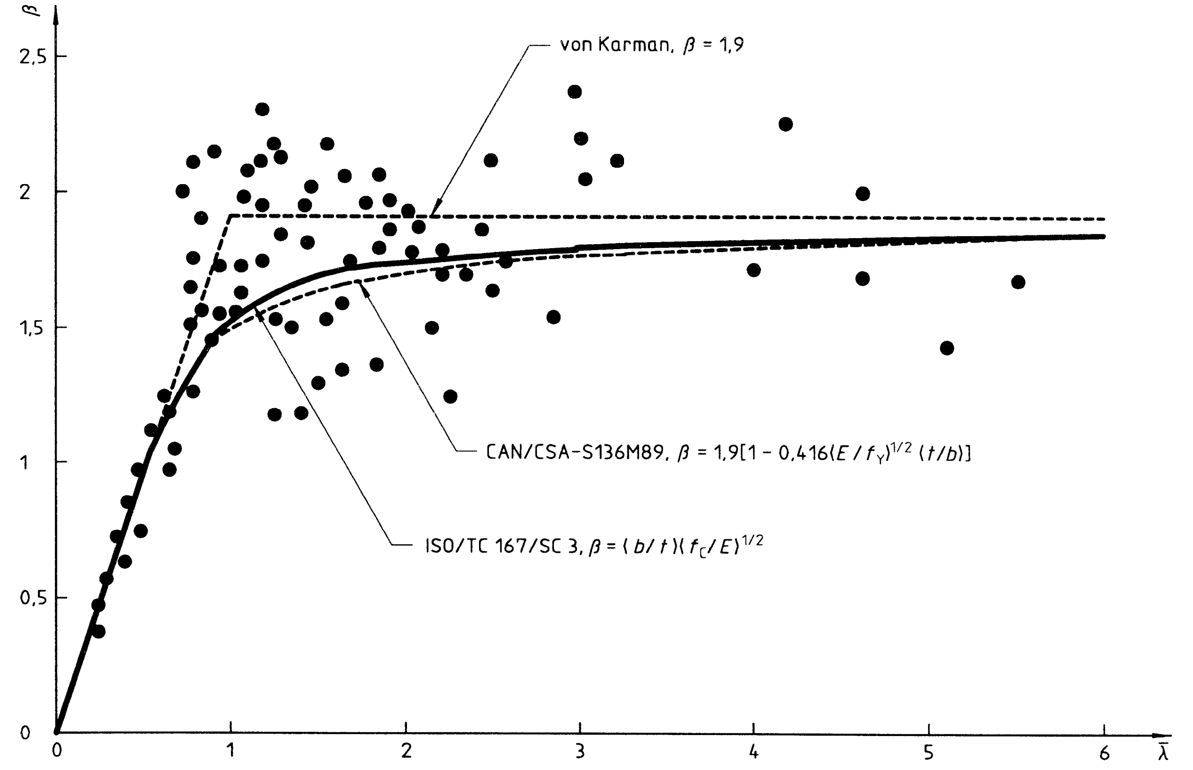


Figure A.5 — Comparison of effective width formulae with test results for steel [41]

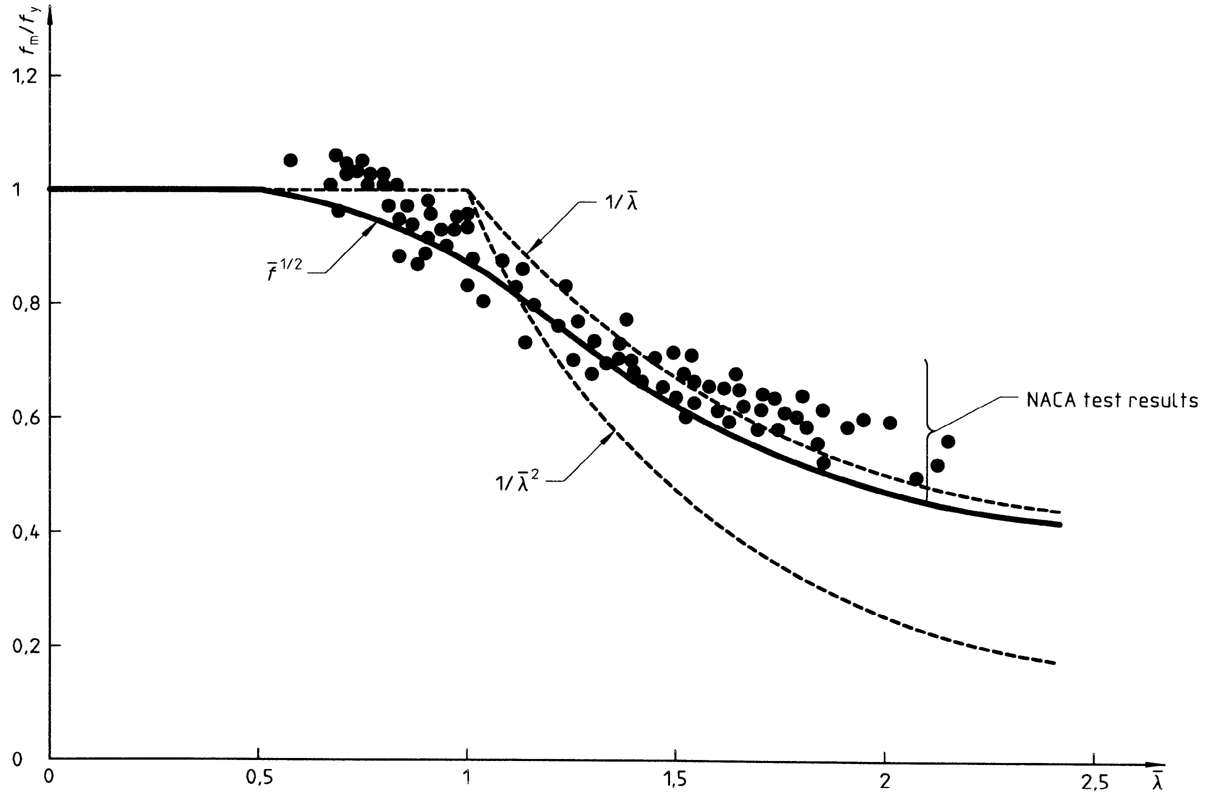


Figure A.6 — Comparison of post-buckled strength formula with test results [54]

A.10.4.2 Post-buckling strength of shear webs [12.4.2]

Shear stress in a panel cannot exceed the yield strength in shear of the base metal of the web, $f_{sy} = 0{,}6f_y$, while the shear flow cannot exceed the ultimate strength in shear of the boundary connections of the web or of any seams in the web. This latter value, identified as v_k, may be controlled by the ultimate shear strength at a weld, $f_{shu}t = 0{,}6f_{hu}t$, or the strength of fillet welds, or by the strength of a line of rivets or bolts, R/s, where R is the resistance of a fastener and s is the spacing of the fasteners.

It is assumed that the shear stress remains uniform across the web up to the critical value, and the total shear force at this stage is given by

$$V_k = htf_{sc} \leqslant hv_k$$

where

- h is the web depth in the direction of the shear force;
- t is the thickness;
- v_k is the shear resistance along the boundaries.

A stiffened web is created by the presence of transverse stiffeners at points of local reaction or applied force, in such a manner that panels are formed, of any aspect ratio, with stiffeners or flanges at all four boundaries. In this case initial buckling due to shear does not precipitate collapse. As the shear force increases, the initially uniform shear stress changes its distribution along the boundaries, the stress at the compression corner remaining at the initial buckling stress while the stress at the tension corner increases until it reaches some limiting condition [46], [48], [73]. At this stage the stress at the boundary is pure shear, with no stress normal to the boundary. The total shear resistance is the integral of the area under the curve of the shear stress distribution, times the thickness. For a hyperbolic stress distribution this integral leads to

$$V_k = (f_{sc}f_{smax})^{1/2}ht$$

where f_{smax} is the maximum shear stress at the tension corner of the web.

Although this expression provides a convenient parallel with the effective stress used for post-buckled plates in compression, it is too conservative. The maximum stress, f_{smax}, cannot exceed the limiting value provided by yielding in shear or by the ultimate strength in shear of welded or riveted connections, v_k/t, but because of a "gusset effect" in the tension corner there is a zone along which the limiting stress is maintained. The capacity of the web in shear then becomes [46]:

$$V_k = [2(f_{sc}v_k/t)^{1/2} - f_{sc}]ht$$

To explain the post-buckling strength of panels with a high aspect ratio [38] showed that there is a change in the orientation of the stress field. The direction of the principal stresses rotates and, while the principal compressive stress remains constant at the value that caused initial buckling, the principal tension stress increases in such a manner as to maintain zero stress normal to the boundaries. When the maximum shear stress reaches the shear yield strength, the shear force on a transverse plane section through the web becomes (figure A.7):

$$V_r = \left(2f_{sc}f_y - f_{sc}^2\right)^{1/2}ht$$

This gives values sufficiently close to those obtained by the earlier formula, for the practical range of web dimensions, that it permits a single expression to be adopted for all panel aspect ratios.

Should the limiting shear strength be equivalent to the yield strength in the base metal, as is true in steel and for annealed aluminium alloys, then, from the viewpoint of the designer, the shear resistance may be considered to be attributable to

a) an effective shear strength:

$$f_{sm} = f_{sy}\left(2\bar{f}^{1/2} - \bar{f}\right)$$

which will be multiplied by the gross web area to give the shear resistance; or:

b) an effective thickness [73]:

$$t' = \left(2\bar{f}^{1/2} - \bar{f}\right)t$$

which will be multiplied by the shear yield strength.

It is to be observed that up to this stage no use is made of the notion of "diagonal tension". With a further increase in the shear force, the yielded zones in the tension corners distort in shear, compelling the flanges to bend. But the flanges have an independent bending strength:

$$M_o = b_f t_f^2 f_y / 4$$

where b_f and t_f are the width and thickness of the flange respectively.

When the flange carries an axial force, N, the fully plastic moment resistance is reduced by the factor $\left[1 - (N/N_p)^2\right]$, where N_p is the fully plastic axial force.

The resistance of the flanges to this imposed bending will cause the development of some "diagonal tension", and thereby make a contribution to the shear resistance given by

$$V = 2\left(M_o t f_y\right)^{1/2}$$

Because of the nature of aluminium construction, where the shear strength is controlled by welds or riveted seams, there may be insufficient shear distortion available to develop any contribution by diagonal tension before rupture at the boundary occurs [29], [48]. In order that this additional capacity be available, the ultimate strength at the boundaries must exceed the yield strength of the basic web metal, as in some welded nonheat-treated alloys.

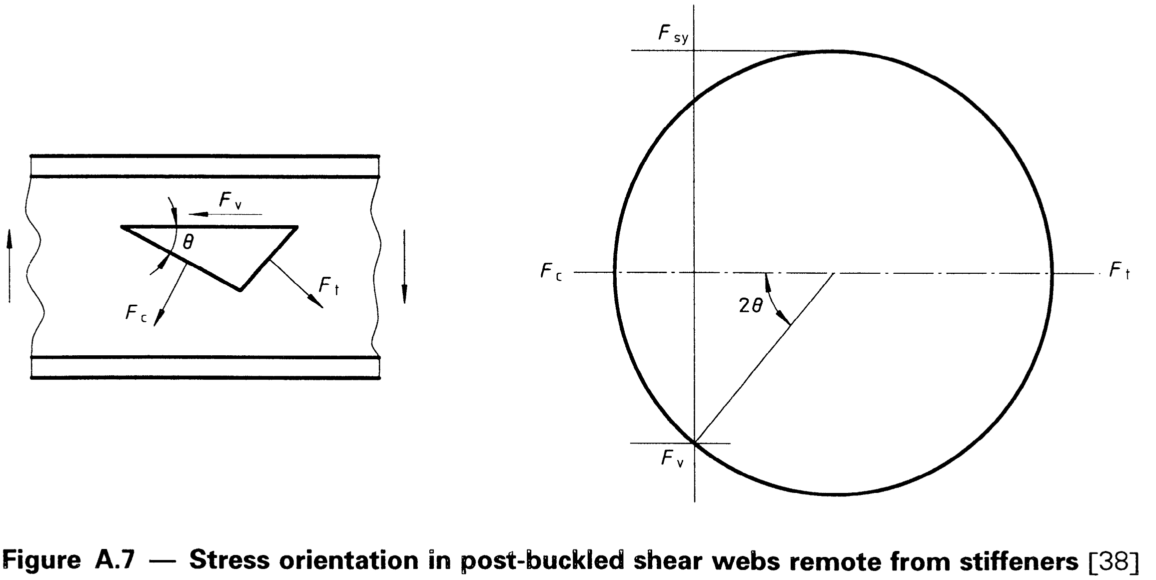


Figure A.7 — Stress orientation in post-buckled shear webs remote from stiffeners [38]

A.10.4.3 Webs with transverse and longitudinal stiffeners [12.4.3]

Where the web is divided in panels of different sizes, the panel with the highest slenderness, at the section of interest, determines the initial buckling stress, f_{sc}, and hence the mean shear stress at the ultimate capacity. This value limits the capacity to transmit shear force across the web and will be used to obtain the total shear resistance at that location.

A.10.4.4 Combined shear and moment [12.4.4]

When a web buckles under the action of combined shear force and bending moment, rather than attempt to apportion the capacity between the two functions, it is more practical to let the web carry the shear force and let the flanges carry the bending moment.

A.10.4.5 Shear web stiffeners [12.4.5]

The stiffeners act in compression when the web is buckled, due to the difference in the shear flows on the two sides of each stiffener. At the limiting capacity, they will carry a maximum axial force given by

$$N = v_k s\bar{f}\left(1 - \bar{f}^{1/2}\right)/\left(1 + \bar{f}^{1/2}\right)$$

where s is the length of the stiffener.

In the case of transverse stiffeners, the force approaches the shear force at the stiffener as $\bar{f}$ diminishes. To accommodate this force, the stiffener is designed as a strut of length equal to the web depth.

For longitudinal stiffeners, the compressive force created in the stiffener after the web buckles, as $\bar{f}$ diminishes, approaches:

$$N = v_k s\bar{f} = (f_{sc} v_k t)^{1/2} s$$

The stiffener is designed as a strut to resist this force, spanning between transverse stiffeners.

A.10.4.6 Web crushing [12.4.6]

Under the direct local action of an applied force or a reaction the failure may be caused by

a) buckling of the web, with an upper limit of yielding along the contact line;

b) local bending of the radiused web to flange junction in formed sheet sections.

Buckling of the web due to a local force is treated in [72]. For a load distributed along a short length, n, the force required to buckle that length of web as a column is added to the value for the local force. Dividing this total force by the nominal area $(n + h)t$ gives the nominal buckling stress, f_e, for which the slenderness is $\lambda = 2h/t$. This value of λ is used to obtain the buckling stress.

The buckling stress is reduced by the presence of bending stresses in the web, and the classical interaction formula is used.

For forces near the ends, the area is reduced linearly, over a length h, to $(n + h/2)t$.

In extruded sections with square corners between the flange and web, the local stress along the line of contact cannot exceed the yield strength.

In formed sheet shapes, the radiused corners cause a serious reduction in local strength.

The expression adopted is that in CAN3-S157-M83 and parallels that used for steel but with some simplifications based on the tests reported in [5], in which the influence of the compressive bending stress on the crushing strength was also shown.

A.10.5 Curved walls [12.5]

A.10.5.1 Tubes in axial compression [12.5.1]

The theoretical value for the elastic buckling stress due to longitudinal compressive force in tubes is too optimistic. Based on work [23], the critical stress is given by an expression which reflects the increasing sensitivity of the buckling stress to imperfections as the ratio r/t increases. Using this expression, a slenderness is obtained which, in conjunction with the yield strength of the alloy, gives the normalized slenderness for entry into the plate buckling formula.

Internal pressure helps to increase the stability of a shell by the action of reducing the imperfections. In these cases reference should be made to the literature.

A.10.5.2 Tubes in radial compression [12.5.2]

An expression to give the critical stress for tubes subjected to radial compressive force is taken from [72], and the slenderness is that for a well-formed cylinder satisfying the usual commercial tolerances. While cylindrical tubes carrying radial compression are not as sensitive to imperfections as when axially loaded, if there is severe ovality a full analysis should be made. On the other hand, the presence of circumferential stiffeners increases the stability, reducing this sensitivity. If advantage is to be taken of this, reference should be made to the literature.

A.10.5.3 Tubes in shear [12.5.3]

Shear, caused by lateral loads or torque, is treated using the expression taken from [72] to give the elastic buckling stress and is converted to a form that can be used with the normalized buckling curves for plates.

A.11 Torsion [clause 13]

It is expected that the full plastic resisting torque can be developed in those sections which rely on St. Venant torsion.

A.12 Bolted and riveted connections [clause 14]

A.12.1 Use of fasteners [14.1]

Design procedures for the use of fasteners apply to steel and aluminium bolts and solid aluminium rivets. Special fasteners and joining techniques find wide use in joining sheet and where access is from only one side. There is no restriction on the use of methods so long as the level of reliability is maintained.

The actual material of a bolt, given that it is of sufficient strength, is only of concern where corrosion might occur. For rivets, the material must be capable of being upset without damaging the parent material.

Pre-loaded bolts, permitting the joint to transfer the force by friction, are not commonly used in aluminium but there is now sufficient experimental work [27] to provide recommendations should there be a need for rigidity under service loads. Surface treatment, such as sanding, is required.

It is preferable that all bolts act in bearing at the ultimate limit state, thus discouraging the use of holes slotted in the direction of loading.

Fasteners in a row loaded longitudinally are not uniformly stressed and, although yielding will largely reduce this non-uniformity by the time the ultimate load is reached, if the joint is unusually long there may be a reduction in the overall capacity. The limits of six fasteners and a length of $15d$ have been shown to give an acceptable performance [32].

A.12.2 Spacing of fasteners [14.2]

The minimum spacing of fasteners is governed by the clearance for bolt heads and tools, build-up of compressive stress generated by cold-formed rivets, and limits to the validity of the design expressions given.

The maximum spacing between fasteners is usually governed by local plate buckling in compression. For fastener rows widely spaced across the direction of force ($s < 1{,}3g$), the plate buckles into multiple half-waves with all edges fixed, which is represented by a slenderness $\lambda = 1{,}3g/t$. For fasteners widely spaced along the direction of force ($s > 1{,}3g$), the plate buckles over the length s with fixed edges, for which $\lambda = 1{,}7s/t$. For staggered patterns of fasteners, the rectangular zone with fixed edges is defined by two pairs of fasteners spaced at $2g$ and $2s$. When $s = g$ this gives a value of $\lambda = 1{,}9s/t$.

Linear relationships are adopted as $s/g \rightarrow 0$ and $g/s \rightarrow 0$, to the limiting values of $1{,}3g/t$ and $1{,}7s/t$ respectively.

A.12.3 Strength of joints [14.3]

A.12.3.1 Bolts and rivets in shear [14.3.1]

Fasteners in shear are designed using the ultimate shear strength of the fastener material. Although the von Mises criterion, which gives a shear yield strength equal to $1/\sqrt{3}$ times the tensile yield strength, does not apply to ultimate strengths of the "engineering" variety, the customary ratio of 0,6 is adopted as it gives conservative values for the shear strength.

A.12.3.2 Bolts and rivets in bearing [14.3.2]

Bearing stress is computed as the stress exerted by the fastener on the wall of the hole, although that particular stress, as such, is never critical. Failure is by the tearing of the material adjacent to the hole and correlates with the ultimate shear strength of the metal [31], [44]. For force directed towards an edge, as the end distance, e, increases, the resistance increases, approaching a constant value after the edge distance exceeds twice the fastener diameter. Calculated on the basis of shearing along the two planes beside the hole leads to the resistance $R = 2f_{su}et$. Using the Tresca criterion, $f_{su} = f_u/2$, gives $R = f_u et$, a convenient expression that has been shown to be conservative.

When force is directed towards an oblique edge, the tear-out force is related to the angle between the edge and the direction of force in such a manner that the resistance changes from a bearing failure to a tensile failure across the net section as the angle changes from 90° to 0°.

Unrestrained lap joints in tension are eccentrically loaded, causing a reduction in strength. This is most pronounced when the plates are of equal thickness, for which case the bearing strength is halved. This value is based on tests for steel sheet reported in [10].

A.12.3.3 Tear-out of bolt and rivet groups ("block shear") [14.3.3]

A.12.3.3.1 [**14.3.3.1**] The expressions giving the resistance to tear-out of bolt and rivet groups are simple extensions of the tear-out model for single fasteners [44] and include the tension failure of the material between the fasteners in line across the direction of stress. In the case of the shear connection to a web this mode of failure is termed "block shear" in steel.

A.12.3.3.2 [**14.3.3.2**] See A.12.3.3.1.

A.12.3.3.3 [**14.3.3.3**] See A.12.3.3.1.

A.12.3.3.4 [**14.3.3.4**] The tear-out failure of a compact group of fasteners subjected to a torque is assumed to occur along a circular shear path which encloses the bolt group, using $f_{su} = f_u/2$. This type of failure has been demonstrated in tests and the treatment is known to be realistic, but there is no body of research data available.

A.12.3.4 Concentric force on group of fasteners [14.3.4]

For a force whose line of action passes through the centroid of a group of fasteners, the distribution of force between the fasteners is assumed to be uniform at the ultimate resistance. The known inequality of the elastic force distribution is largely eliminated by the non-linear response before failure. This is subject to the limitations to the total number of fasteners and the length in 14.1.6.

A.12.3.5 Eccentric force on groups of fasteners [14.3.5]

A.12.3.5.1 Elastic behaviour [14.3.5.1]

In the case of an eccentric force acting on a group of fasteners, the analysis in the elastic range assumes that the fasteners behave linearly and that the connected material is rigid. A group of n fasteners subjected to a force, N, applied at an eccentricity, e, is considered to carry a force, N, through the centroid of the group, and a moment, Ne.

If each fastener has a spring constant, k, the group will translate bodily in the direction of the force by a distance:

$$\delta = N/nk$$

The group will rotate through an angle given by

$$\theta = Ne/kI_o$$

where

$$I_o = \Sigma r_i^2$$

r_i is the distance from the centroid to the ith fastener.

Combining this translation and rotation, the centre of rotation of the group is at a distance, c, from the centroid such that $\delta = c\theta$, giving

$$c = I_o/ne$$

The maximum force on a fastener is then

$$R_{max} = \frac{N(e + c)d_{max}}{\left(I_o + nc^2\right)}$$

where d_{max} is the distance from the centre of rotation to the furthest fastener.

Using $c = I_o/ne$, this becomes

$$R_{max} = \left(\frac{N}{n}\right)\left(\frac{d_{max}}{c}\right)$$

A.12.3.5.2 Characteristic static resistance [14.3.5.2]

At the ultimate resistance, the fasteners are all assumed to be fully plastic, and, although it is not strictly valid, it is convenient to use the same effective centre of rotation for the fully plastic condition, which leads to [46]:

$$N(e + c) = \Sigma R_r d_i$$

where

R_r is the resistance of a fastener;

d_i is the distance from the centre of rotation to the ith fastener.

The analysis used in [25] for the non-linear behaviour of steel bolts gives similar predictions.

A.12.3.6 Friction-type joints [14.3.6]

Procedures for the design of joints using high-tension bolts to provide sufficient friction force to resist the loads are drawn from [27].

A.12.4 Fasteners in tension [14.4]

A bolt in tension fails across the net section at the thread. The constrained region of the net section inhibits necking and leads to an ultimate tensile strength closer to the "true" value than to the "engineering" value. Some recognition of this is seen in the use of a "stressed area" in excess of the true net area.

A.13 Welded connections [clause 15]

For design purposes, the nominal stressed area of a weld bead is the plate thickness for butt welds, and the shortest distance through the nominal isosceles inscribed triangle for fillet welds, with no regard paid to bead reinforcement. Butt weld strengths are, in fact, established for weld beads with all reinforcement removed.

Inspection after fabrication will establish whether welds of the required size and quality have been provided (ISO 10042).

A.13.1 Mechanical properties [15.2]

Table A.1 gives mechanical properties of the HAZ, and table A.2 gives the ultimate strength of the weld bead. The suggested values are discussed under 6.4.2.3.

A.13.2 Butt joints [15.3]

The tensile values given in the tables are used to establish the tension strength of butt joints, unless this exceeds the yield strength of the unaffected base metal [27].

When a welded joint, subjected to compression, is constrained to remain straight, the limiting value in tension is used [27]. This condition occurs at the end of a compression member [55] or a T-joint carrying a moment [47].

Where instability can occur, it is reasonable to limit the compressive stress to the yield strength in the HAZ, as the local reduction in elastic modulus after yielding often precipitates local buckling.

A.13.3 Fillet welds [15.4]

A.13.3.1 Concentrically loaded fillet welds [15.4.2]

Fillet welds, when concentrically loaded, have customarily been treated as though they were in shear, regardless of the direction of loading. It is now recognized that the strength varies with the direction, but the question remains as to how the strength relates to the tensile properties. Shear yield stress is limited to $f_y/\sqrt{3}$, using von Mises yield criterion, and for any combination of normal and shear stresses:

$$\left(f_t^2 + 3f_s^2\right) < f_y^2$$

While this criterion is recognized as a reasonably valid condition for yielding, it cannot be expected to apply to the "engineering" values used for the ultimate strengths, for a number of reasons.

- The true ultimate strength on the reduced area is not, in general, available.
- The "engineering" ultimate stress is artificial.
- Shear failure in fillet welds involves no "necking", so the full area remains active, with the result that the ultimate shear strength can exceed 75 % of the "engineering" tensile ultimate.

No consistent correlation between the properties of the base metal and the welded joint has been found, nor can the ultimate strength of a fillet weld be related to the original yield strength of any of the parts of the weld. There is no generally accepted method for computing the actual stress in a fillet weld, so tests on welds themselves are needed to provide the design stresses, and the measured values provide the most appropriate basis for the relationships between them.

For the case of a force applied parallel to the weld direction, the true shear strength of the weld bead is used. Because different researchers have tested fillet welds loaded in different directions [70], [47], [55] the conclusions about the influence of the direction of force are not always in agreement. There have been a number of attempts to explain why the weld loaded in direction Z is stronger than that loaded in direction Y, in figure 17, but tests have consistently demonstrated it to be true, when the throat is in tension. On the other hand, tests have also suggested that, for the case in direction Z, the strength in compression is lower than in tension.

The resistance per unit length of a fillet weld is given by

$$v_k = kaf_{wu}$$

where

a is the throat size;

f_{wu} is the ultimate tensile strength of the weld bead.

Values for the factor k, for the three directions of loading, are given in table A.8, and are compared with the values used in [2], [75] and [27] for aluminium, and [18].

Table A.8 — Values for factor k

Direction of loading	Factor k for		
	X	Y	Z
ISO/TC 167/SC 3	0,6	0,7	0,8
AA	0,6	0,6	0,8
CAN3-S157-M83	0,6	0,6	0,85
CISC (steel)	0,67	0,95	0,95
ECCS	0,6	0,85	0,85

For oblique load directions, a simple spherical relationship between the three components is suggested. Another method is to calculate the stresses on the weld throat due to the orthogonal components of the force and combine them using the von Mises criterion for yielding. As discussed above, as the true stresses are not readily calculated and von Mises is not valid for ultimate strengths, it would appear more reasonable to make use of the actual measured strengths of fillet welds.

A.13.3.2 Eccentrically loaded fillet welds [15.4.3]

A.13.3.2.1 Moment about Y-axis [15.4.3.1]

A.13.3.2.1.1 Elastic behaviour [15.4.3.1.1]

An elastic analysis for eccentrically loaded fillet welds is required when fatigue is a consideration, in order to compute the maximum force per unit length in the weld. The method used for this analysis is then extended to give the limiting static load when the welds are fully plastic [47].

The procedure follows that used for eccentrically loaded groups of fasteners (14.3.5.2), replacing the number of bolts, n, by the weld length, H, and the value of I_0 by the polar moment of inertia of the weld pattern, I_p, using the throat thickness a.

The weld rotates about a point at a distance c from the centroid, on a line normal to the line of action of the force, passing through the centroid, on the side away from the force, given by

$$c = I_p / Hae$$

The maximum force per unit length of the weld is given by

$$v_{max} = Na(e + c)d_{max} / \left(I_p + Hac^2\right)$$

Using $c = I_p / Hae$ this becomes

$$v_{max} = (N/H)(d_{max}/c)$$

A.13.3.2.1.2 Characteristic static resistance [15.4.3.1.2]

To obtain the characteristic resistance, N_k, the same centre of rotation is assumed for plastic behaviour, although this does not strictly satisfy equilibrium. The applied moment is then $N_k(e + c)$ and the ultimate resisting moment is $v_k\Sigma(L_i d_i)$ giving

$$N_k = v_k\Sigma(L_i d_i) / (e + c)$$

where

L_i is the length of the ith element of the weld;

d_i is the distance from the centre of rotation to the centre of the ith element;

v_k is the resistance per unit length of the weld.

Comparing this simple method with the computer-generated values used for the non-linear behaviour of steel fillet welds [16] shows good agreement.

An alternative method is to show that a statically fully plastic model can be found which resists the applied force. This gives a safe design but no information about the actual capacity of the joint.

A.13.3.2.2 Moment about Z-axis [15.4.3.2]

For eccentrically loaded double-fillet-welded T-joints, it is necessary to consider different strengths in tension and compression [49]. If f_{ty} and f_{cy} are the yield strengths in a rectangular section, $L \times t$, the fully plastic moment is given by

$$M_o = \left(tL^2/2\right) f_{cy} f_{ty} / (f_{cy} + f_{ty})$$

For welded T-joints, the limiting stresses are provided by the strengths in tension and compression of the HAZ, the tension strength of the fillet welds and the yield strength of the base metal. The tensile strength of the fillet welds is reduced by the presence of the shear force, and use is made of the formula for combined forces (15.4.2.2).

A.13.4 Flare groove welds [15.5]

Welds made between round bars or at formed corners rely on the penetration achieved for their strength. As this varies with the radius of the surface, it is not easy to predict with confidence the final effective throat size, and it is required that the specified size be demonstrated in practice.

A.13.5 Slot and plug welds [15.6]

Welds formed by filling a slot or hole with weld metal are subject to shrinkage cracks and are not permitted to carry calculated stress.

Annex B
(informative)

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