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Ioannis Vayas · John Ermopoulos
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Design of Steel Structures to Eurocodes

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Preface

Buildings characterize urban areas and are related to the personal, social and professional activities of people. The selection of the appropriate materials for its structural elements (reinforced concrete, steel, aluminum, wood, masonry), depends on the characteristics of the building and the design criteria, such as economic, aesthetic, functional, execution time, as well as the conditions of soil quality and seismicity of the construction area. Steel, as a main structural material, is used in all countries, in a different by country extent, depending on the local conditions and the existing tradition in the construction methods. Steel buildings may be distinguished in single and multi storey. Single storey steel buildings are mainly erected for industrial, commercial, warehousing and sports applications. Multi storey constructions are mainly used for residential or office purposes.

The design and fabrication of buildings are performed following rules provided in specifications and Codes. During the last decades, an extended program of common Codes for all European countries was developed, called Eurocodes, covering both design and fabrication issues, in order to facilitate mobility of construction companies, design offices and engineers in the area of the European Community and beyond it. In addition the cooperation between authorities and technical organizations and personnel, coming from different countries, should become easier.

This book presents the rules for the design of steel buildings according to the above Eurocodes, covering the structure as a whole, as well as the design of individual structural members and connections. The presentation is supplemented by many numerical examples. Specific sections of the book are dedicated to the conceptual design, the fabrication and erection phases and the quality requirements. Rules for the seismic design, when required, are also included. The text is organized in 9 chapters. Chapters 1-5 deal with the methods of analysis, the limit states of design and the resistances of cross-sections, members and connections, while chapters 6-8 are related to the conceptual design of single and multi storey buildings as well as to the fabrication methods and the quality control. Chapter 9 includes numerical applications of the design rules in the form of 52 design examples.

Chapter 1 presents the bases of design, in the frame of Eurocodes, the actions applied to building structures, the load combinations for the various limit states of design, as well as the main steel properties and the steel fabrication methods.

Chapter 2 deals with the models and methods of structural analysis, in combination with the structural imperfections and the cross-section classification according to their compactness.

Chapter 3 discusses the cross-sections resistances, when subjected to axial and shear forces, bending or torsion moments and to combinations of the above.

Chapter 4 presents the members design and more specifically the design of members sensitive to instability phenomena, such as flexural, torsional and lateral-torsional buckling. A particular section is devoted to composite beams.

Chapter 5 refers to the design of connections and joints executed by bolting or welding, including beam to column connections in frame structures.

Chapter 6 discusses alternative configurations to be considered during the conceptual design phase of different types of single storey buildings. The design of crane supporting beams is discussed in a special section.

Chapter 7 gives information about the structural elements and systems of multi storey buildings, especially those ensuring their overall stability along the height, as well as about the alternative configurations that could be applied.

Chapter 8 refers to the fabrication and erection procedures, as well as the related quality requirements and the quality control methods. The procedures for bolting, welding and surface protection are included.

Chapter 9 presents fifty two representative numerical examples, based on the design rules for the verification of cross-sections and members, subjected to the usual types of loading, the verification of bolted and welded connections, as well as for specific items such as hollow sections' joints, uniform built-up compression members or column bases. The calculation steps are directly related in the text with the corresponding paragraphs of Eurocodes.

The book is addressed to the structural engineering students, to young engineers working in the field of design or construction of steel buildings, as well as to engineers not familiar with the regulations of Eurocodes.

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Contents

Symbols

Effort was made for the symbols to be in accordance with those used by the Eurocodes. A single symbol is used for those quantities where different symbols are used by different Eurocodes.

General symbols for geometric properties

b	width
d	depth
h, H	height
t	thickness
l, L	length, span

General symbols for mechanical properties

A	area
I	second moment of area (moment of inertia)
S	first moment of area (static moment)
W	cross-section modulus

General symbols for internal forces and moments

M	bending moment
M_T	torsional moment
N	axial force
V	shear force

General symbols for stresses

σ	direct (normal) stress
τ	shear stress

Indexes

a	structural steel
add	additional
b	beam
bat	batten
B	buffer

bear	bearing
c	concrete, compression, column, crane
ch	chord
cr	critical value
d	design value, diagonal
dur	durability
E	action effect
eff	effective
el	elastic
eq	equivalent
f	flange
f_o	upper flange
f_u	lower flange
fat	fatigue
G	permanent
H	horizontal
inf	lower value
k	characteristic value
L	longitudinal, longterm
max	maximum value
min	minimum value
nom	nominal value
o	top, opening
ov	over-strength
p	plate panel zone
pay	payload
pl	plate, plastic
r	rail
R	resistance
s	reinforcement, skewing
S	shrinkage, short
sa	steel + reinforcement
ser	serviceability
sup	upper value
sur	surface
t	tension
T	torsional, transverse
tot	total
u	ultimate, limit value, bottom
V	vertical
w	web, warping, weld
y	yield
I	first order
II	second order

Axes

x	longitudinal axis of member
y	major principal axis of cross-section
z	minor principal axis of cross-section

Operators

Δ	difference
δ	variation

Latin small letters

a	length, thickness of a fillet weld, distance between wheels, spacing, clearance, skewing angle
a_g	peak ground acceleration
b	width, free gap
b_{eff}	effective width
b_{fo}	width of top flange of steel girder
b_{fu}	width of bottom flange of steel girder
b_o	half distance between webs
b_r	width of a rail, width of a rib
b_s	spacing between ribs
c	outstand flange width, concrete cover of reinforcement/shear connectors, smeared spring constant, hole clearance
c_e	exposure factor
c_f	wind coefficient, force coefficient for the structure or elements
c_{min}	minimum value of concrete cover
c_{nom}	nominal value of concrete cover
c_{pe}	pressure coefficients for external pressure
c_{pi}	pressure coefficients for internal pressure
$c_s\ c_d$	structural factor
c_ϕ, c_θ	stiffness of rotational spring
d	differential, diameter, shank diameter of shear connector, length of diagonal, grain diameter, thickness of concrete slab, depth of cross-section
$d_{head,sc}$	head diameter of shear connector
d	hole diameter
d_r	design interstorey drift
e	eccentricity, length of a link
e_D	edge distance of shear connectors from steel flange
e_E	edge distance of shear connectors from concrete slab
e_L	spacing of shear connectors in longitudinal direction, length of a long link
e_s	length of a short link
e_T	spacing of shear connectors in transverse direction
e	imperfection, initial, bow
e_1, e_2	end/edge distance of bolts
f	reduction factor, stress, skewing angle coefficient, source to object distance

f_{cd}	design compression strength of concrete
f_{ck}	characteristic compression strength of concrete
f_u	tensile strength
f_{ub}	tensile strength of bolts
f_{vw}	shear strength of welds
f_y	yield stress of steel
f_{yd}	design yield strength of steel
f_{yk}	characteristic yield stress of structural steel
$f_{y,w}$	yield stress of web
g	permanent load, acceleration of gravity, gap, length of a haunch
g_a	self-weight of steel girder
g_c	self-weight of concrete
h	height, magnitude of weld imperfection
h_c	height of concrete slab
h_p	height of a trapezoidal sheet
h_r	height of rail
h_w	height of web
h_o	notional size, height of a web opening
i	index, radius of gyration
i_M	polar radius of gyration in respect to shear center
i_p	polar radius of gyration
k	spring constant, constant for relationship between yield strength and grain size, floor stiffness
k_1, k_2	reduction coefficients for concrete strength
k_σ	plate buckling coefficient
k_τ	shear buckling coefficient
l	length
l_{eff}	effective length
l_k	buckling length
l_T	buckling length in respect to torsion
l_y	effective loaded length
m	mass, distributed moment
m_{Ny}	non-dimensional strong axis bending resistance allowing for axial force
$m_{N.y.w}$	non-dimensional resistance of web for strong axis bending allowing for axial force
m_{Nz}	non-dimensional weak axis bending resistance allowing for axial force
m_u	non-dimensional bending moment for angles strong axis
m_v	non-dimensional bending moment for angles weak axis, number of single wheel drives
m_y	non-dimensional bending moment, strong axis
m_{yf}	non-dimensional resistance of flanges for strong axis bending
m_{yw}	non-dimensional resistance of web(s) for strong axis bending
m_z	non-dimesional bending moment, weak axis
m_{zf}	non-dimensional resistance of flanges for weak axis bending
m_{zw}	non-dimensional resistance of web(s) for weak axis bending

n	non-dimensional axial force, number, number of, modular ratio of concrete, distribution coefficient
n_f	non-dimensional axial resistance of both flanges
n_L	modular ratio depending on the type of loading
n_w	non-dimensional axial resistance of web(s)
n_o	number of openings on beam's web
n_1, n_2	non-dimensional resistance of web(s) for weak axis bending, distribution factors
p	uniformly distributed load, pitch between bolt holes
p_R	probability of exceedance
p_1, p_2	spacing of bolts, parallel/perpendicular to force
q	uniformly distributed load, behavior factor
q_b	basic wind pressure
q_{fk}	uniformly distributed imposed load on buildings
q_p	peak velocity pressure
$q_{u,kin}$	collapse load of kinematic theorem
$q_{u,stat}$	collapse load of static theorem
r	radius
s	snow load, length in lattice girders, spacing of staggered bolt holes, spacing
s_k	snow load, characteristic value
s_o	spacing between web openings
t	thickness
t_f	thickness of flange
t_{fo}	thickness of top flange of steel girder
t_{fu}	thickness of bottom flange of steel girder
t_w	thickness of web
u	perimeter, displacements, cross-section major axis of bending for angles
v	loading speed, deformation in y-direction, cross-section minor axis of bending for angles, travelling speed of a crane
v_b	basic wind velocity
v_L	longitudinal shear flow
$v_{L,Ed}$	longitudinal shear flow, design value
$v_{L,Rd}$	longitudinal shear flow, design resistance
v_{Mt}	shear flow due to torsional moments
w	width, deformation in z-direction, deflection
w_M	deflection due to moments
w_V	deflection due to shear forces
x_{pl}	depth of plastic neutral axis

Greek small letters

α	aspect ratio of panel, imperfection factor, amplification factor
α_{crit} ,α_{cr}	critical load multiplier
α_f	ratio of the area of both flanges to the total area of the cross-section

α_h, α_m	reduction factors for sway imperfection
α_{LT}	imperfection factor for lateral torsional buckling
α_{pl}	shape factor of a cross-section
α_t	coefficient of thermal expansion
$\alpha_{ult.k}$	load multiplier to reach the characteristic resistance
α_w	ratio of the web area to the total area of the cross-section
β	buckling length coefficient
β_w	correlation factor for welds
γ	safety factor, specific weight, sliding angle
γ_A	partial safety factor of accidental actions
γ_{AE}	partial safety factor of seismic actions
γ_c	partial safety factor for concrete
γ_f, γ_F	partial safety factors for actions
γ_{Ff}	partial safety factor for fatigue stresses variation
γ_G	partial safety factor of permanent actions
γ_m	partial safety factor for a material property
γ_{M}	partial safety factor for resistance
$\gamma_{\mathrm{M}f}$	partial safety factor for fatigue resistance
γ_{M0}	partial safety factor for yield
γ_{M1}	partial safety factor for stability
γ_{M2}	partial safety factor for fracture and connections
γ_{M3}	partial safety factor for slip
γ_Q	partial safety factor of variable actions
γ_{Rd}	partial safety factor for resistance
γ_s	partial safety factor for reinforcement
γ_v	partial safety factors for shear connectors
γ_I	importance factor
δ	deflection, Dischinger coefficient
δ_{pay}	deflection due to payload
ε	strain, coefficient depending on f_y
ε_y	yield strain
ε_u	ultimate strain
θ	rotation, angle of twist, inter-story sensitivity coefficient
θ_p	rotation of a plastic hinge
κ	curvature
λ	slenderness, damage equivalent coefficient, skewing forces coefficient, overlapping percentage
λ_1	reference slenderness
$\bar{\lambda}$	relative slenderness for flexural buckling
$\bar{\lambda}_{LT}$	relative slenderness for lateral torsional buckling
$\bar{\lambda}_{op}$	relative slenderness for out-of-plane buckling
$\bar{\lambda}_p$	relative slenderness for plate buckling
$\bar{\lambda}_w$	relative slenderness for shear buckling
$\bar{\lambda}_y$	relative slenderness for major axis buckling
$\bar{\lambda}_z$	relative slenderness for minor axis buckling

μ	slip factor, ductility index, efficiency factor for built-up members
μ_i	snow shape coefficient
ν	Poisson ratio, reduction factor
ξ	damping ratio
ρ	density, reduction factor for plate buckling, reduction factor for presence of shear, strut index, warping rigidity
ρ_s	reinforcement ratio
σ	direct stress
σ_a	stress in structural steel
σ_c	stress in concrete
σ_{cr}	critical buckling stress
$\sigma_{cr,p}$	critical stress for plate buckling
$\sigma_{cr.FT}$	critical flexural-torsional buckling stress
$\sigma_{cr.T}$	critical torsional buckling stress
σ_s	stress in reinforcement
σ_{true}	true stress
σ_w	stress in web
τ	shear stress
τ_{cr}	critical shear buckling stress
τ_{Ed}	design shear stress
τ_f	flange shear stress
τ_w	web shear stress
ϕ	angle, dynamic factor
ϕ	initial sway imperfection
χ	buckling reduction factor
χ_c	reduction factor for column buckling
χ_{LT}	reduction factor for lateral-torsional buckling
χ_{op}	reduction factor for lateral or lateral-torsional buckling
χ_w	reduction factor for shear buckling
χ_y	reduction factor for major axis buckling
χ_z	reduction factor for minor axis buckling
ψ	stress ratio, end moments, ratio combination factor of one action with other actions
ψ_0	basic value of combination factor
ψ_1	frequent value of combination factor
ψ_2	quasi permanent value of combination factor
ω	warping function
ω_0	natural circular frequency

Capital letters

A	cross-section area, gross section area, accidental action
A_d	cross-section area of a diagonal
A_E	seismic action
A_{eff}	effective cross-section area
A_f	flange area of cross-section

A_{net} net section area at holes
A_p gross area of plate
A_{ref} reference area for wind force
A_s stress area of bolts
A_u minimum cross-section area after fracture
A_v shear area
A_w web area of cross-section, throat area of weld
A_0 initial cross-section area, area enclosed by the middle line of a hollow section
B warping bimoment
B_{Ed} design warping moment
$B_{el.Rd}$ elastic bimoment resistance
$B_{p.Rd}$ punching resistance of bolts
$B_{pl.Rd}$ plastic bimoment resistance
C concrete, creep of concrete, wind load factor, spring constant
C_d rotational spring, limiting design value of the effects of actions in SLS
C_e exposure coefficient
C_m equivalent uniform moment factors
C_t thermal coefficient
D maximum acceptable relative displacement, diameter, compressive force
Δh height variation
Δl elongation, contraction
ΔT_M linear temperature difference
ΔT_p temperature difference between structural parts
ΔT_u uniform temperature difference
E modulus of elasticity
E_a modulus of elasticity of structural steel
E_c modulus of elasticity of concrete
$E_{c,28}$ modulus of elasticity of concrete at 28 days
E_{cm} modulus of elasticity of concrete – mean value
E_d design value for effect of actions
$E_{d,dst}$ design value for effects of destabilizing actions
$E_{d,stb}$ design value for effects of stabilizing actions
E_D absorbed hysteretic energy
E_s modulus of elasticity of reinforcement
E_t tangent modulus of steel
F force
$F_{b,Rd}$ bearing resistance of bolts
F_{cr},F_{crit} critical concentrated load, buckling load
F_d design value of an action, force of a diagonal
F_{Ed} imposed design load
$F_{f.Rd}$ bottom flange strength under concentrated load
F_k characteristic value of an action
$F_{p.C}$ preloading force of bolts, design shear force of bolts
$F_{s.Rd}$ slip resistance of bolts
F_T force on a T-stub

$F_{t.Rd}$	tension resistance of bolts
$F_{v.Rd}$	shear resistance of bolts
F_W	wind force, force on weld
G	weight, shear modulus, permanent action
G_1	self-weight of the structure
G_2	self-weight of non-structural elements
G_{inf}	permanent actions with favorable effects
G_{sup}	permanent actions with unfavorable effects
H	horizontal force, lateral force, altitude
HB	Brinell hardness
H_{Ed}	design horizontal loading
I	second moment of area (moment of inertia),
I_{eff}	second moment of area of effective part
I_{net}	second moment of area of net section
I_p	second moment of area of plate, polar second moment of area of a stiffener
I_t	torsion constant
I_w	warping constant
I_y	second moment of area around strong axis
I_z	second moment of area around weak axis
J	creep function, impact energy, torsion constant
K	stiffness , drive force
K_G	geometric stiffness matrix
L	length, span, longitudinal force
L_e	distance between zero moments
L_{eff}	effective length for resistance to concentrated forces
L_f	influence length
L_Φ	determinant length
M	bending moment, mass
$M_{a,el,Rd}$	elastic design moment resistance of steel girder
$M_{c,Rd}$	design bending resistance
M_{Ed}	design moment
M_{el}	elastic moment of a cross-section
$M_{el,Rd}$	elastic design moment resistance
$M_{f,Rd}$	design bending resistance of cross-section consisting of the flanges only
$M_{N,pl,Rd}$	design bending resistance of cross-section allowing for axial forces
$M_{N.y.Rd}$	design strong axis bending resistance allowing for axial forces
$M_{N.z.Rd}$	design weak axis bending resistance allowing for axial forces
M_{pl} ,M_p	plastic moment of a cross-section
M_{pN}	plastic moment allowing for axial forces
$M_{pl,Rd}$	design plastic bending resistance
$M_{pl.u.Rd}$	strong axis plastic design bending resistance for angles
$M_{pl.v.Rd}$	weak axis plastic design bending resistance for angles
$M_{pl.V.Rd}$	design plastic bending resistance allowing for shear forces
$M_{p.link}$	bending plastic resistance of a link
$M_{pl.y.Rd}$	plastic design bending resistance, strong axis
$M_{pl.z.Rd}$	plastic design bending resistance, weak axis

M_r	torque for tightening of bolts
M_R	bending resistance
M_{Rd}	design bending resistance
M_{sh}	primary shrinkage moment
$M_{T,Ed}$	design torsional moment
M_w	bi-moment
M_x, M_t	torsional moment
M_{xp}	primary torsional moment
M_{xs}	secondary torsional moment
$M_{y.Ed}$	design strong axis moment
$M_{z.Ed}$	design weak axis moment
M_I	moment from first order theory
M_{II}	moment from second order theory
N	axial force
$N_{b,Rd}$	design buckling resistance
N_c	axial force in concrete
$N_{c,el}$	force in concrete at elastic resistance of steel girder
$N_{c,f}$	force in concrete for full shear connection
N_{cr}	Euler buckling load, axial force at cracking of concrete
$N_{cr,y}$	critical buckling forces for major axis
$N_{cr,z}$	critical buckling forces for minor axis
$N_{c,Rd}$	design resistance to compression
N_E	critical Euler buckling load
N_{Ed}	design axial force
$N_{Ed.G}$	design axial force due to the non-seismic loads
$N_{net.Rd}$	design yielding resistance at net section
N_{pl}, N_p	plastic axial force
$N_{pl.Rd}$	plastic design resistance force
N_R	resistance to axial force
N_s	axial force in reinforcement
N_{sh}	primary shrinkage axial force
$N_{t,Rd}$	design resistance to tension
$N_{u,Rd}$	design ultimate resistance to tension for sections with holes
P	load, force, prestressing
P_e	limit elastic force
P_p	limit plastic force
P_{Rd}	shear resistance of shear connectors
P_u	ultimate force
Q	variable action, imposed load on buildings
Q_k	variable load, concentrated load, characteristic value
Q_{max}	maximum value of the characteristic vertical wheel load
$Q_{r.min}$	minimum load per wheel of the unloaded crane
R	resistance
R_d	design resistance
R_{fy}	resistance of a dissipative member
RH	relative humidity

S	static moment (first moment of area), snow load, fatigue class of a crane, force applied by the guidance means of a crane wheel
Sa	blast cleaning class
$S_{a,d}$	design response spectrum, design spectral acceleration
S_B	spring constant of a buffer
S_e	elastic response spectrum, elastic spectral acceleration
S_j	rotation stiffness of a joint
S_v	shear stiffness
T	temperature, vibration period, transverse force, total torsional moment
T_{Ed}	design torsion moment
T_L	design life
T_R	return period of an event
T_t	St Venant torsion moment
$T_{t.pl.Rd}$	plastic Saint Venant torsion resistance
T_w	warping torsion moment
U	class of crane related to the total number of cycles
V	shear force, vertical load, total potential
$V_{b,Rd}$	shear buckling resistance
$V_{bf,Rd}$	design shear resistance-contribution of the flange
$V_{bw,Rd}$	design shear resistance-contribution of the web
V_L	force due to longitudinal shear
$V_{p.link}$	plastic shear resistance of a link
V_{Ed}	design shear force, design vertical loading
$V_{pl,Rd}$	plastic shear resistance
V_{Rd}	design shear resistance
W	section modulus, wind load
W_{eff}	elastic section modulus of effective cross-section
W_{el}	elastic section modulus
W_{pl}	plastic section modulus
$W_{pl.u}$	strong axis plastic section modulus for angles
$W_{pl.v}$	weak axis plastic section modulus for angles
X	material property
Z	through thickness property
Δ	erection or manufacturing tolerance
$\Delta\delta$	deflection variation
$\Delta\sigma$	normal stress variation
$\Delta\sigma_c$	fatigue resistance against normal stress variation
$\Delta\tau$	shear stress variation
$\Delta\tau_c$	fatigue resistance against shear stress variation
ΣQ_r	sum of all wheel vertical loads of a loaded crane
Φ	Coefficient to determine the reduction factor χ, diameter of bars
Ω	ratio N_{Rd}/N_{Ed} or M_{Rd}/M_{Ed}

1 Basis of Design

Abstract. This chapter introduces the objectives of steel structures and their main fields of application that include almost all types of works in the construction sector, listing the most important advantages for the basic material in terms of mechanical properties, the high prefabrication, the easiness of connection between elements, but also the "after construction" benefits, like recyclability, durability, or easiness for strengthening and repair. It makes reference to the Eurocodes and other specifications on which this book is based and more specifically to the parts and structure of Eurocode 3 that specifies the design of steel structures. It then gives the main types of actions and provides information on the determination of the most important ones for building structures like imposed loads, wind, snow, temperature or earthquake. The ultimate and serviceability limit states are defined and the combinations of actions, together with the relevant partial safety and combination factors. Finally, the main mechanical properties of steel and the structural steel grades are presented, with some basic information on the microstructure of steel and common steel making processes that help understand the source of attaining the specified mechanical, physical and chemical properties.

1.1 Introduction

The use of iron dates back to some thousands BC, most probably in the Far East. The first written witness of workmanship in the West, more than 3000 years ago can be found in Odyssey of Homer [1.1]. The poet, referring to the moment of the blindness of the Cyclop says that "I was reminded of the loud hiss that comes from a great axe or adze when a smith plunges it into cold water – to temper it and give strength to the iron. That is how the Cyclop's eye hissed round the olive stake". In many ancient marble tombstones, specifications for workmanship, quality assurance and quality control of various metal alloys, forerunners of the modern standards closely associated with the use of steel, were found.

With the use of coke, pure carbon, rather than charcoal as a fuel in a blast furnace that started 1709 from A. Darby but extended after 1750 in England a high temperature could be reached that melted completely the iron which was subsequently cast in special molds, and is therefore called **cast iron**, in order to take its final form. **Cast iron** contains carbon (C) more than 1% by weight and due to its high C-content

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has high resistance to compression but small in tension. Later in the 1780s H. Cort developed puddling furnaces where C was burning from above of the melt while by puddling the melt impurities could be removed. This process leads to the production of the higher quality **wrought iron** that has equal resistance to compression and tension and allowed the manufacture of the first standardized shapes.

This early iron was rather brittle due to its high carbon content. Consequently, the carbon had to be lowered to produce steel that has by definition a carbon content lower than 2% and may be forged both in the cold and warm state. The first production of steel was done in converters developed in England around 1850 by H. Bessemer through oxidation, with air being blown through the melt, a process that raised the temperature and kept the iron mass molten. The mass production of steel started actually a little later, when C. Siemens developed in Germany and P. Martin applied in France an open hearth furnace which allowed the oxidation of carbon until the wished carbon content was achieved after which the process was terminated. The above were acid processes for which clay linings were used to plate the converter. For raw material with high phosphorus content, as in Lorraine France, the alkaline Thomas process developed in 1878 was applied, where the converter was plated with dolomite linings that extract phosphorus from the melt [1.2].

Steel is today of two types: the one that is produced by the *basic oxygen* process and the *electric steel.* In the first process, the reduction of liquid pig iron from the blast furnace to steel is done by blown oxygen, not air as by Bessemer, in a converter called a ladle. The process is autogenous since oxidation generates heat in the ladle. The second process uses only solid scrap as input material that is melt by an electric arc forming between the charged material and graphite electrodes that are lowered into the electric arc furnace. Such furnaces range in size from a few to several hundred tons allowing the decentralized production in mini-mills.

By a secondary steelmaking with strict control of metallurgy, higher grades of steel and steel with special properties may be produced. Today's yearly steel production is around 1600 million tons, half of which in China. This corresponds to a compact column with the cross-section dimensions of a football stadium, 50x100 m, and a height of 40 km.

The first applications of iron as structural material in buildings started around the mid-19th century. Since the material was cast iron with high compression and low tension resistance, it was mainly applied for compression elements, like columns or curved beams supporting floors or roofs. This happened in Britain around 1850 for reasons of fire safety, especially in weaving mills where the work was done in an oily atmosphere under the candle lights and the wooden construction was gradually replaced by metal construction. It was the time of the industrial revolution, where the construction of industrial buildings started, with load carrying masonry walls at the exterior and internal frames composed of metallic beams and columns.

At the same time, with the expansion of the railways started the necessity for roofs of railway stations and other roofs to cover large spaces, like markets, exhibition centers, shipyards etc. The structural systems for these roofs were mostly plane trusses, plane arches or domes, but in particular three hinged arches due to the fact that they are isostatic and therefore simpler for hand calculations and not susceptible to differential settlements or temperature variations. With the use of horizontal string, a reduction of the arch height and accordingly of the volume was made possible, resulting in a further reduction of costs. At the same time buildings of metal and glass were built, the most characteristic of which is Crystal Palace in London with 70000 m^2 plan area, which was built 1851 at a construction time of 10 months.

With the rapid industrial development after 1920 the requirements on larger spans and small erection times grew, leading to increasing use of truss girders of different types, like in bridge construction. The susceptibility of large span plane trusses to lateral buckling, but also the increased need to cover spaces irregular in plan brought the invention after 1940 of space frames which constitute 3D truss systems composed of typified nodes connected to the bars by special connections.

The need for high rise buildings started towards the ends of the 19^{th} century due to population growth and high land prices in the large cities. Then a remarkable construction activity developed, especially in the USA, with the erection of constantly higher buildings up to today's skyscrapers. The construction of skyscrapers continues uninterrupted, the relay being given to the high populated countries of East Asia and especially China. In Shanghai where over 15 buildings with more than 170 m height have been built in the last 5 years, the visitor has the impression that high rise buildings grow in modern times from the ground like mushrooms. Skyscrapers not only cover urban or other needs but they define also the architectural picture of the city (Fig. 1.1).

Fig. 1.1. Skyline of Shanghai, China

Steel construction was and is still the standard solution for certain applications like industrial buildings. Steel finds also its way in non-industrial projects, like in residential and office buildings, a market with predominance of reinforced concrete. This is mainly due to an overall reduction of costs in steel solutions as a result of:

- Application of fast track construction, using metal deck composite construction with pumped concrete that lowered the construction time.
- The possibility to provide clear spans in the range of 10-18 m, so satisfying requirements for maximum flexibility.
- A diversity of architectural treatment and the provision of structural solutions for free form design.
- Lower prices for basic materials and application of high strength steel.
- Availability of smart connection techniques.
- The introduction of fire engineering approaches and a move towards lightweight fire protection systems such as boards, sprays and intumescent coatings that resulted in a massive drop of fire resistance costs.

However, the application of steel structures is not limited to building, or bridge structures. In fact, steel has the largest range of application compared to all other construction materials.

Other application areas, sometimes without competition from other materials, are:

- Lattice towers and masts, whether guyed or not, for communication or electric transmission.
- Silos and tanks.
- Chimneys.
- Towers for wind power stations.
- Cranes and crane supporting girders.
- Scaffolds.
- Pallet racking systems.
- Offshore structures.
- Locks and sluice gates.

Besides steel structures where steel is the only structural material, there exist also composite structures where steel acts compositely with reinforced concrete. Applications of composite structures may be found primarily in multi-story buildings or bridges.

Steel has some material properties that make it the most flexible, possibly the most suitable, building material such as:

- High resistance to weight ratio that leads to lighter structural weights, possibility to bridge larger spans, lower foundation loads etc.
- Homogenous industrially produced material with predefined and guaranteed properties.
- Ductility that allows redistribution of forces, especially in case of accidental loads and unexpected situations.
- High connectivity that allows a speedy erection.
- Durability, provided sufficient maintenance is guaranteed.
- Recyclability and low carbon equivalent.
- Easiness for strengthening and repair.
- Combination of low inertial forces, due to low weight, and ductility that makes it optimal in seismic resistant structures.
- High degree of prefabrication.

1.2 Codes and Specifications

The present book makes reference to the Eurocodes as design Standards, which were developed after an effort of almost 40 years and are currently used exclusively in the countries of the European Union. Although structural steel design is covered by Eurocode 3 only, all Eurocodes have to be consulted in any specific project, whether it is a building or other type of project. The Structural Eurocode programme comprises currently the following standards generally consisting of a number of Parts:

EN 1990 Eurocode 0: Basis of structural design [1.3]
EN 1991 Eurocode 1: Actions on structures [1.4]
EN 1992 Eurocode 2: Design of concrete structures [1.5]
EN 1993 Eurocode 3: Design of steel structures [1.6]

EN 1994 Eurocode 4: Design of composite steel and concrete structures [1.7]
EN 1995 Eurocode 5: Design of timber structures [1.8]
EN 1996 Eurocode 6: Design of masonry structures [1.9]
EN 1997 Eurocode 7: Geotechnical design [1.10]
EN 1998 Eurocode 8: Design of structures for earthquake resistance [1.11]
EN 1999 Eurocode 9: Design of aluminium structures [1.12]

The first two Eurocodes are of general application for design, EN 1990 describing the requirements for safety, serviceability and durability of structures, EN 1991 providing the actions ("loads") on structures. All other Eurocodes related to superstructures (i.e. all except EN 1997) concern specific construction materials (concrete, steel, composite steel and concrete etc.). A special case is EN 1998 that includes both a generic part on seismic actions, as well as specific parts related to types of structures with relevant construction materials.

Due to the large field of application of steel structures, Eurocode 3 is the most extended Eurocode with the largest number of parts and pages. It is structured such that it has a core and a periphery. The core includes generic rules, such as strength, stability, fatigue or fire design, applicable to all types of structures and contains following documents:

- EN 1993-1-1: General rules and rules for buildings [1.13]
- EN 1993-1-2: Structural fire design [1.14]
- EN 1993-1-3: Supplementary rules for cold-formed thin gauge members and sheeting [1.15]
- EN 1993-1-4: Supplementary rules for Stainless steels [1.16]
- EN 1993-1-5: Plated structural elements [1.17]
- EN 1993-1-6: Strength and Stability of Shell Structures [1.18]
- EN 1993-1-7: Strength and stability of planar plated structures subjected to out of plane loading [1.19]
- EN 1993-1-8: Design of joints [1.20]
- EN 1993-1-9: Fatigue [1.21]
- EN 1993-1-10: Selection of materials for fracture toughness and through-thickness properties [1.22]
- EN 1993-1-11: Design of structures with tension components [1.23]
- EN 1993-1-12: Additional rules for the extension of EN 1993 up to steel grades S 700 [1.24]
- EN 1993-1-13: Beams with large web openings (under preparation)

The periphery refers to specific types of constructions such as buildings, bridges, towers etc. It makes reference to the core, while any part of the latter is used by more than one parts of the periphery. For example, part 1.9 of the core on fatigue is referenced by part 2 on bridges and part 6 on crane supporting structures of the periphery. The parts referring to specific types of constructions are the following:

- EN 1993-2: Steel Bridges [1.25]
- EN 1993-3-1: Towers, masts and chimneys – Towers and masts [1.26]
- EN 1993-3-2: Towers, masts and chimneys – Chimneys [1.27]
- EN 1993-4-1: Silos [1.28]

- EN 1993-4-2: Tanks [1.29]
- EN 1993-4-3: Pipelines [1.30]
- EN 1993-5: Piling [1.31]
- EN 1993-6: Crane supporting structures [1.32]

It is noted that the National Standards implement the various Eurocodes by means of a relevant **National Annex.** This Annex contains country specific data, e.g. for climatic or seismic actions. It also provides design values of some parameters where the Eurocodes give recommended values. All these data are referred as nationally determined parameters. This book uses the recommended values of those parameters as provided in the body of the Code.

Finally concerning Codes and Specifications, it should be mentioned that besides Eurocode 3 there are several Euronorms, ENs, that cover different issues such as material properties, fabrication, erection, quality control etc. Among these, special mention need to be made to EN 1090-2 "Execution of steel structures and aluminium structures" an important document concerning construction [1.33].

However, the correct application of all Specifications including the Eurocodes is associated to the following assumptions:

- The selection of the structural system and its design will be performed by appropriately educated and experienced personnel.
- Execution of the work is carried out by appropriately skilled personnel with adequate equipment.
- All activities and their products, whether in the office or on site are subjected to sufficient inspection and quality control.
- The construction materials and products are of the quality and are used as in the relevant specifications.
- The work will be operated according to its design assumptions.
- The work will be maintained properly.

1.3 Actions

Civil engineering works are subjected during their design life to various loads due to operation, exposure to climatic conditions or other influences. The structure should be accordingly designed such that with sufficient degree of reliability and in an economic way it can withstand all actions (loads) to which will be possibly exposed during its design life. The design life depends mainly on the type of structure and is usually fixed as 100 years for bridges or infrastructure projects, 50 years for buildings, 15 to 30 years for agricultural structures, 10 years for temporary structures etc. [1.3].

Actions on structures are specified in EN 1991, which describes their qualitative and quantitative dimension as a result of systematic measurements or long-standing meteorological observations. Their proposed values have a predefined small probability of exceedance and are called characteristic values. For some type of actions different values are proposed country by country in the relevant National Annex due to different climatologic conditions. The probability of exceedance of a certain action during the design life of a structure is connected to the return period of a certain action event through the relation:

$$T_R = -\frac{T_L}{\ln(1-p_R)} \tag{1.1}$$

where:

T_R is the return period of the event
T_L is the design life and
p_R is the probability of exceedance.

For example, for a probability of exceedance $p_R = 10\%$ in a design life of $T_L = 50$ years, or annual probability 0.2%, the probability of exceedance is $T_R = 475$ years.

Actions are distinguished by EN 1990 [1.3] in relation to their duration, magnitude and probability of occurrence in:

- **Permanent (G)**, which have small variation in time. They can further on distinguished into:
  - $\mathbf{G}_1$ Self weight of the structure and
  - $\mathbf{G}_2$ Self weight of non-structural elements
- **Variable (Q)**, which vary considerably in time. For buildings they include:
  - **Q** Imposed loads on buildings
  - **S** Snow loads
  - **W** Wind loads
  - **T** Temperature variations
  - Settlements etc.
- **Accidental (A)**, with short duration and significant magnitude, with small probability of occurrence (e.g. fire, impact, column loss)
- **Seismic ($\mathbf{A}_E$)**, which develop in seismic zones during an earthquake ground motion

Further on the actions are distinguished in:

- **Direct**, that are forces (loads) applied to the structure, e.g. self-weight, imposed loads, wind etc.
- **Indirect**, that develop due to imposed deformations or accelerations, e.g. during temperature changes, uneven settlements, or earthquake.

1.3.1 Permanent actions G

Permanent actions include the self-weight of the structure and of non-structural elements such as roofing, surfacing, partition walls, facades, suspended ceilings, thermal insulation etc. Self-weights are represented by a single characteristic value and are based on the nominal values for dimensions and densities as provided in [1.34], Annex A. For the structural elements self-weights are automatically calculated by the analysis software, possibly increased by a small percent, e.g. 5%, to account for gusset plates, bolts etc. For manufactured elements data provided by the manufacturer are adopted. Self-weights of partitions are usually transformed from line loads into uniformly distributed area loads (UDL) and added to the G_2-load.

1.3.2 Imposed loads on buildings Q

Imposed loads on buildings arise mainly from occupancy and include persons, furniture or movable objects of normal use, but not weights of heavy equipment. Imposed loads are modelled as UDL (q) for general effect or concentrated loads (Q) for local effects, not acting simultaneously, the values of which depend on the category of use. Table 1.1 shows categories and recommended values. The National Annex to [1.34] may define other values.

Table 1.1. Recommended values for imposed loads on buildings [1.34]

Category	*Use*	*Example*	q_k [kN/m^2]	Q_k [kN]
		Residential, social, commercial and administration areas		
A	Domestic and residential			
Floors			2.0	2.0
Stairs			2.0	2.0
Balconies			2.5	2.0
B	Office		3.0	4.5
C	Areas where people congregate	C1 schools, restaurants, cafes	3.0	4.0
		C2 theaters, churches conference rooms	4.0	4.0
		C3 hotels, hospitals, administration	5.0	4.0
		C4 dance halls, gymnastic rooms	5.0	7.0
		C5	5.0	4.5
D	Shopping	D1 general retail	4.0	4.0
		D2 department stores	5.0	7.0
		Storage and industrial use		
E1	Storage	general and books	7.5	7.0
E2	Industrial use			
		Parking areas		
F	Light vehicles $\leqslant$ 30 kN	Garages	2.5	20
G	Medium vehicles $\leqslant$ 160 kN	Access routes, delivery zones	5.0	90
		Roofs		
H	Non accessible, except maintenance		0.4	1.0
I	Accessible, occupancy of categories A to D		As for categories A to D	
K	Accessible for special services, e.g. helicopter landing			20-60

1.3.3 Snow loads S

Snow loads S are variable loads acting on roofs with characteristic values s_k [kN/m^2] on ground derived from snow maps provided in the National Annexes to [1.35]. Snow loads on the roofs are usually taken as UDL loads s (Fig. 1.2), determined from:

$$s = \mu_i C_e C_t s_k \tag{1.2}$$

where:

$\mu_i, i = 1,2$ is the snow shape coefficient that depends on the slope of the roof.
C_e is an exposure coefficient with recommended values 0.8, 1.0 or 1.2.
C_t is a thermal coefficient with values below 1.0 if the roof is heated.

For multi-span roofs, discontinuous roofs or at obstructions, drifted snow arrangements are considered, in which s varies linearly along the roof (Fig. 1.2).

Exceptional snow loads, with 2times the load as recommended value, or snow drifts may be considered for the accidental design situation.

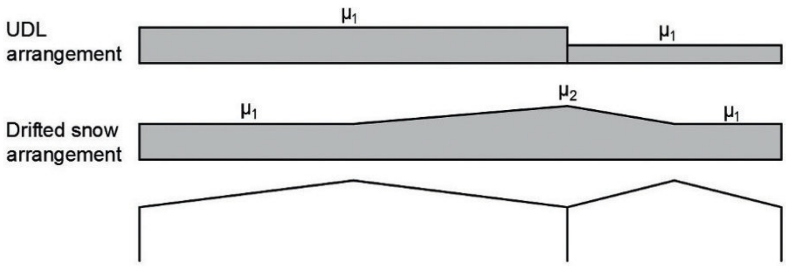


Fig. 1.2. Snow loads on roofs

1.3.4 Wind loads W

Wind loads are very important for many types of steel structures and often constitute the main horizontal load. For not too flexible structures dynamic influences are low and wind loads may be considered as of static nature. Wind loads are defined in EN 1991-1-4 [1.36] as pressures or suctions acting normal to the surface. The most important parameters that define wind loads are the wind velocity which depends on the geographic location, the terrain roughness and orography or the height above ground, but also the shape of the structure.

The relationship between basic wind pressure q_b and basic wind velocity v_b is expressed by eq. (1.3).

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \tag{1.3}$$

where:

$\rho = 1.25$ kg/m^3 is the density of the air and the index b refers to basic values.

The basic wind velocity v_b is defined in [1.36] as the characteristic 10 minutes mean wind velocity at 10 m above ground in open country terrain with low vegetation (terrain category II).

The peak velocity pressure at height z, $q_p(z)$, is associated with the basic wind pressure by eq. (1.4):

$$q_p(z) = c_e(z) \cdot q_b \tag{1.4}$$

where:

$c_e(z)$ is the exposure factor provided by the Code [1.36] as a function of z and the terrain category.

Wind pressures acting at external (e) and internal (i) surfaces are determined from (Fig. 1.3):

$$w_{e \text{ or } i} = q_p(z) \cdot (c_{pe} \text{ or } c_{pi}) \tag{1.5}$$

where:

$q_p(z)$ is the peak velocity pressure at height z and
c_{pe} or c_{pi} the pressure coefficients for external or internal pressure.

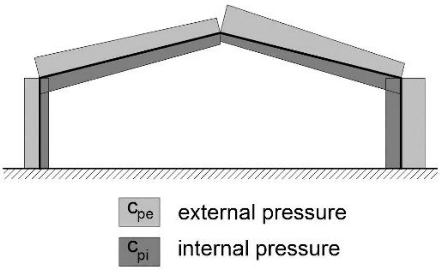


Fig. 1.3. Wind pressures on buildings

Net pressures are determined by algebraic addition of internal and external pressures $c_{pe} + c_{pi}$ or by net pressure coefficients $c_{p,net}$ that are also provided for some cases in the Code [1.36].

As an alternative to pressures, wind forces on the whole structure or its elements may be determined from:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z) \cdot A_{\text{ref}} \tag{1.6}$$

where:

$c_s c_d$ is the structural factor, as the product of the size and the dynamic factor
c_f is the force coefficient for the structure or elements and
A_{ref} the corresponding reference area.

EN 1991-1-4 [1.36] provides methods to determine coefficients, factors etc. for various typologies, geometries and works, such as buildings, canopies, spheres, lattice structures, flags or bridges, including dynamic effects.

1.3.5 Temperature variations T

Temperature variations act not as loads on structures but as indirect thermal actions. This is due to the fact that they induce deformations, forces and moments that do not solely depend on the magnitude of the variation but also on the conditions of constraint. On an example of a single bar, it can be realized that a temperature rise or fall results in expansion or contraction, i.e. deformations, if the bar is free to move, or alternatively compression or tension forces if the bar is constrained at its ends. Temperature variations in buildings are prescribed in EN 1991-1-5 [1.37]. They may be taken into account as uniform temperature differences ΔT_u, as linearly varying temperatures ΔT_M or as temperature differences ΔT_p between different structural parts. Thermal actions may be taken into account as actions in normal buildings that are protected by thermal insulation. However, for exposed steel structures they must always be taken into consideration, at least by the component ΔT_u that expresses temperature changes between winter/summer and day/night.

1.3.6 Accidental actions A

Fire or other unexpected situations constitute accidental actions. Fire actions are covered by current regulations as accidental action [1.14], but other unexpected situations are not. The importance of their consideration in design of buildings has been recognized after spectacular accidents, like the collapse of the 22-storey Ronan Point apartment, UK due to domestic gas explosion, of the 9-storey office building in Oklahoma, USA due to truck-bomb attack, or of the World Trade Center twin towers, USA due to airplane collision. Since then robustness and progressive collapse has become a major issue in design and research. Values of specified accidental actions, like internal explosions or collisions are provided in EN 1991-1-7 [1.38]. However, it is not always possible to anticipate any kind of accidents that might endanger the integrity of a building or other type of structure. For that reason, the current trend for buildings is to consider situations like column loss that may be the result of accidental or unexpected actions and provide design concepts to cope with [1.39], [1.40] and so evolved the requirement for provide structural robustness.

1.3.7 Seismic actions A_E

Seismic actions develop in structures during earthquakes, where important ground accelerations occur. They are prescribed in EN 1998-1 [1.41] and have to be considered for structures in seismic regions. Seismic actions are not fixed predefined loads but inertial forces of magnitude that depends on several parameters like ground acceleration, soil conditions, system's flexibility, structural ductility and damping. Most of these parameters may be influenced by the design, so that reference is made to seismic design for structures in seismic regions. Seismic forces are determined from site dependent response spectra. A response spectrum gives the acceleration of the structure as a function of its vibration period. The spectral acceleration multiplied by the mass provides the seismic forces. Fig. 1.4 shows a response spectrum for elastic structural behavior, in which the spectral acceleration is divided by the peak ground acceleration (PGA).

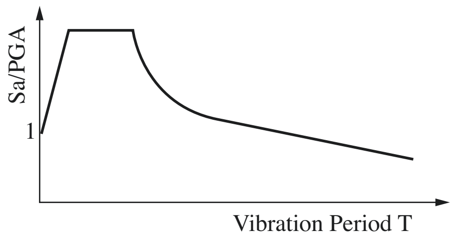


Fig. 1.4. Elastic response spectrum

It may be seen that, depending on the period and therefore on the stiffness, the spectral acceleration of the structure may be many times larger than the PGA and that it becomes smaller only for very flexible structures. The spectral acceleration becomes smaller also if the structure yields and behaves inelastic. Consequently, for structures in regions of high seismicity where PGA can reach values approaching the gravity acceleration, g, ductile design is anticipated where the structure behaves non-linearly and withstands lower seismic forces. However, non-linear behavior may lead to structural damage that is not always tolerated by all types of structures, such as vital facilities, power plants etc., so that the concept of performance based seismic design has evolved which introduces structural performance levels associated to certain probability of exceedance.

1.4 Limit States and combinations of actions

1.4.1 General

A steel structure has to comply during its design life with certain basic requirements that refer to structural resistance, serviceability and durability and are met by appropriate design, production, execution and use. Concerning the design this is based on consideration of two limit states, the ultimate and serviceability limit states [1.3].

Ultimate limit states (ULS) are associated with the safety of people and of the structure and refer to:

- EQU: Loss of static equilibrium of the structure or parts of it, regarding them as a rigid body.
- STR: Failure by collapse or excessive deformation of the superstructure or its members and more specifically to:
  i. Resistance of cross-sections and connections
  ii. Stability of members
- FAT: Failure caused by fatigue.
- GEO: Failure or excessive deformation of the foundation and the ground.

Serviceability limit states (SLS) concern the functioning of the structure under normal use, the comfort of people and the structural appearance and are associated with:

- Deformations.
- Vibrations.

In limit states verifications design values are considered. The design values of actions are defined as:

$$F_d = \gamma_f \cdot \psi \cdot F_k \tag{1.7}$$

where:

$F_d =$ the design value of the action.
$F_k =$ the characteristic value of the action.
$\gamma_f =$ the partial safety factor of the action which takes into account the possibility of unfavorable deviations of the action values from the characteristic values.
$\psi =$ combination factor of the action with other action; is either 1.0 or ψ_0, ψ_1, ψ_2.

However, verifications are not made in practice by direct comparison between design values and limit values of actions. Actions result in internal forces and moments, deformations and vibrations in buildings that are characterized as action effects and are evaluated by appropriate structural analysis. The design values of the effects of one action are given by:

$$E_d = \gamma_{Sd} \cdot E\left[\gamma_f \cdot \psi \cdot F_k\right] \tag{1.8}$$

where:

$\gamma_{Sd} =$ the partial safety factor that takes into account uncertainties in modeling the actions and in modeling the structure in analysis.

Usually factors γ_{Sd} and γ_f are merged together to a single partial safety factor:

$$\gamma_F = \gamma_{Sd} \cdot \gamma_f \tag{1.9}$$

The effects of actions are then determined from:

$$E_d = E\left[\gamma_F \cdot \psi \cdot F_k\right] \tag{1.10}$$

Following the classification of actions presented in (1.3) in respect to their duration, partial safety factors are distinguished in:

γ_G for permanent actions
γ_Q for variable actions
γ_A for accidental actions
γ_{AE} for seismic actions

Furthermore two values of the safety factors for permanent actions $\gamma_{G,\text{inf}}$ and $\gamma_{G,\text{sup}}$ are used, depending on whether they produce favorable or unfavorable effects.

For a linear structural response, the analysis may be performed with the characteristic action values and the design values of the action effects determined by multiplication with the safety factors and the combination values, i.e. the action effects are given by:

$$E_d = \gamma_F \cdot \psi \cdot E\left[F_k\right] \tag{1.11}$$

This has an important implication in design when regarding combinations of actions. For linear structural response analysis is made for each individual action separately and the combination refers to the resulting actions. However, for nonlinear response the design values of the actions are combined and analysis is made for each combination.

The design resistances are similarly determined from:

$$R_d = \frac{1}{\gamma_{Rd}} \cdot R\left[\frac{X_k}{\gamma_m}\right] \tag{1.12}$$

where:

$X_k =$ the characteristic value of a material property.
$\gamma_{Rd} =$ partial safety factor covering uncertainties in modeling the resistances.
$\gamma_m =$ partial safety factor of the material.

As for the actions, factors γ_{Rd} and γ_m of resistances are usually merged together to a single partial safety factor:

$$\gamma_M = \gamma_{Rd} \cdot \gamma_m \tag{1.13}$$

The design resistances are then determined from the relevant characteristic values:

$$R_d = \frac{R_k}{\gamma_M} \tag{1.14}$$

1.4.2 Ultimate Limit States (ULS)

The design format for the limit state of static equilibrium (EQU) may be written as:

$$E_{d,\mathrm{dst}} \leqslant E_{d,\mathrm{stb}} \tag{1.15}$$

where:

$E_{d,\mathrm{dst}} =$ the design value of the effects of destabilizing actions.
$E_{d,\mathrm{stb}} =$ the design value of the effects of stabilizing actions.

The design format for the limit state of collapse or excessive deformation (STR and GEO) may be written as:

$$E_d \leqslant R_d \tag{1.16}$$

where:

$E_d =$ the design value of the effects of actions, like internal forces or moments.
$R_d =$ the design value of the corresponding resistances.

The effects of individual actions are combined to form load cases to take into account their simultaneous presence. Three combination types are distinguished: basic, accidental and seismic. In the basic combinations one variable action is considered as leading action, the others being accompanying actions. The leading action in the accidental combination is the accidental action itself, while variable actions are introduced with their combination values and multiplied by the relevant factors ψ. The combinations at ULS other than fatigue are presented in Table 1.2. For the basic combinations at ULS EN 1990 [1.3] offers two alternative expressions to eq. (1.17). Recommended values for safety and combination factors are given in Tables 1.3 and 1.4. The final values for each country can be found in the corresponding National Annex.

Table 1.2. Combinations of actions at ULS

Basic combinations (EQU/STR/GEO)

$$\sum_{J\geq 1} \gamma_{Gj,\text{sup}} \cdot G_{kj,\text{sup}} + \sum_{j\geq 1} \gamma_{Gj,\text{inf}} \cdot G_{kj,\text{inf}} + \gamma_{Q1} \cdot Q_{k1} + \sum_{i>1} \gamma_{Qi} \cdot \psi_{01} \cdot Q_{ki} \quad (1.17)$$

Accidental combinations (A)

$$\sum_{J\geq 1} G_{kj,\text{sup}} + \sum_{j\geq 1} G_{kj,\text{inf}} + A_d + (\psi_{1,1} \text{ or } \psi_{1,2}) \cdot Q_{k1} + \sum_{i>1} \psi_{2,i} \cdot Q_{ki} \quad (1.18)$$

Seismic combinations (A_E)

$$\sum_{j\geq 1} G_{kj} + \gamma_I \cdot A_{Ed} + \sum_{i\geq 1} \psi_{2,1} \cdot Q_{i,i} \quad (1.19)$$

Notation

$+$ does not mean summation but "combination with"
Σ means "the combined effect of"
G_{sup} are permanent actions with unfavorable effects
G_{inf} are permanent actions with favorable effects
Q_1 is the leading variable action
Q_{ki} are the accompanying variable actions
A_d is the accidental action
A_{Ed} is the seismic action
γ_I is the importance factor

Table 1.3. Recommended values of safety factors

Action - situation		*Effect*	
		Unfavorable	*Favorable*
Permanent STR/GEO	G	$\gamma_{G,\text{sup}} = 1.35$	$\gamma_{G,\text{inf}} = 1.0$
Permanent EQU	G	$\gamma_{G,\text{sup}} = 1.10$	$\gamma_{G,\text{inf}} = 0.90$
Variable STR/GEO/EQU	Q	$\gamma_{Q,\text{sup}} = 1.5$	$\gamma_{Q,\text{inf}} = 0.0$

1.4.3 Serviceability Limit States (SLS)

The design format for serviceability limit states may be written as:

$$E_d \leq C_d \quad (1.20)$$

where:

E_d is the design value of the effects of actions, like deflections or frequencies
C_d is the corresponding limiting design value.

Three combinations of actions associated to different verifications are considered at SLS as presented in Table 1.5. It may be seen that the safety factors are taken equal to unity, 1.0. The recommended values of the combination factors ψ_0, ψ_1, ψ_2 for buildings are given in Table 1.4.

Table 1.4. Recommended values of combination factors

Actions	ψ_0	ψ_1	ψ_2
Imposed loads on buildings (Categories, Table 1.1)	0.7	0.5	0.3
Category A	0.7	0.5	0.3
Category B	0.7	0.7	0.6
Category C	0.7	0.7	0.6
Category D	1.0	0.9	0.8
Category E	0.7	0.7	0.6
Category F	0.7	0.5	0.3
Category G	0	0	0
Category H			
Snow loads on buildings in member states			
except Finland, Iceland, Norway, Sweden	0.7	0.5	0.2
for sites located at altitudes H > 1000 m	0.7	0.5	0.2
for sites located at altitudes H ⩽ 1000 m	0.5	0.2	0
Wind loads on buildings	0.6	0.2	0
Temperature in buildings	0.6	0.5	0

Table 1.5. Combinations of actions at SLS

Characteristic combination

$$\sum_{j\geq 1} G_{kj} + Q_{k,1} + \sum_{i>1} \psi_{0,1} \cdot Q_{k,i} \tag{1.21}$$

Frequent combination

$$\sum_{j\geq 1} G_{kj} + P_k + \psi_{1.1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{1.22}$$

Quasi permanent combination

$$\sum_{j\geq 1} G_{kj} + \sum_{i\geq 1} \psi_{2,1} \cdot Q_{k,i} \tag{1.23}$$

Notation

See Table 1.2

1.5 Properties of steel

1.5.1 General

The mechanical properties of steel are of importance for design and construction. The usual properties such as the modulus of elasticity, the yield stress or the tensile strength are used in design calculations for any type of structure. However, there are heavy duty steel structures, such as bridges, offshore structures or similar, that are subjected to fatigue loading, exposed to severe environmental conditions or very low temperatures where toughness and through thickness properties are of equal importance for construction and service. Other properties are of physical nature and concern durability or resistance to high temperatures. For the above reasons, as the requirements become more demanding it is necessary to produce steel of special quality to cope with the client's orders. Such steels are produced by a combination of methods that include alloying, thermal treatment, mechanical treatment or special rolling. It is therefore necessary for the structural steel designer to have some basic knowledge not only on the mechanical properties, but also on steel micro-structure and the ways it is produced [1.42], [1.43].

1.5.2 Mechanical properties of steel

1.5.2.1 Stress – strain curve

The basic material behavior is expressed by the stress-strain curve of steel, Fig. 1.5. This curve is determined experimentally by the tension test that may be performed in a universal testing machine. The tension test is fully described in the relevant EN or ISO documents, such as [1.44]. This refers to the shape of the test specimen, its position in the cross-section, the loading speed as well as other test parameters that depend on the type of structural element, the material thickness and other data. The tension test is performed under deformation control, in order to determine the complete curve including the unloading branch. During the test the applied force P and the elongation Δl within the central part of the specimen with initial length L_0 is recorded, while the initial cross-section area of the specimen is equal to A_0. The resulting engineering stress and strain are determined from:

$$\sigma = \frac{P}{A_0} \tag{1.24}$$

$$\varepsilon = \frac{\Delta l}{L_0} \tag{1.25}$$

The above values are called engineering stress and strain, due to the fact that they are determined on the basis of the initial specimen dimensions. However, during the test the cross-section changes in the neck region, especially near fracture loads, so that true stress and strain may be calculated on the basis of the actual dimensions. These values are determined from:

$$\sigma_{\text{true}} = \sigma \cdot (1 + \varepsilon) \tag{1.26}$$

$$\varepsilon_{\text{true}} = \ln(1 + \varepsilon) \tag{1.27}$$

The engineering curve provides important quantities that characterize the steel grade and are used in design, such as:

- the yield strength f_y. This is conventionally defined as the stress level at which after full unloading the permanent strain is 0.2%.
- the corresponding yield strain $\varepsilon_y = f_y/E$
- the tensile strength f_u at maximum loading
- the corresponding ultimate strain ε_u
- the modulus of elasticity E, which is the initial slope of the curve.

The engineering $\sigma - \varepsilon$-diagram indicates the three parts of material behavior: elastic, yielding and strain hardening. The reversible part is the elastic part. Ductility of the material is defined by the ductility index $\mu = \varepsilon_u/\varepsilon_y$ that is the ratio between the ultimate and the yield strain.

Engineering $\sigma - \varepsilon$-diagrams are used in practical design. True $\sigma - \varepsilon$-diagrams are mostly used in refined non-linear FEM (Finite Element)-calculations with strain and stress concentrations in small zones, where areas of fractures or cracks should be determined numerically.

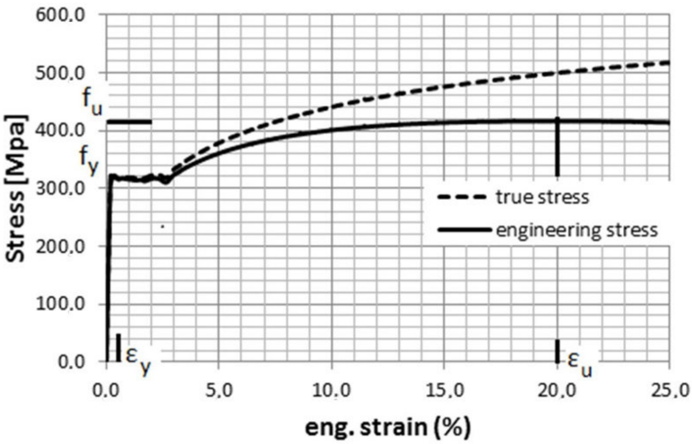


Fig. 1.5. Stress – strain curves of steel from tensile test

1.5.2.2 Material toughness

Material toughness indicates ductile or brittle failure and becomes more and more important as steel is used in heavy duty applications. Toughness is expressed as the energy absorbed during the impact of a small specimen that leads to fracture. The usual impact test is the Charpy V-notched test, with following typified dimensions:

- Specimen simply supported with a clear span 40 mm.
- Square specimen cross-section with dimensions 10×10 mm.
- Typified V-shaped notch.
- Impact speed 5.75 m/sec.

The test is performed at different temperatures and the impact energy is recorded at each one of them. Fig. 1.6 illustrates the material toughness of two steels as a function of the temperature. It can be seen that three regions may be distinguished.

- An upper shelf region with ductile failure.
- A lower shelf region with brittle failure
- A transition zone between ductile and brittle failure.

1.5.2.3 Hardness

Hardness is a property that concerns mostly other engineering application, not structural steels. The most usual test is the Brinell test in which a ball shaped steel indenter

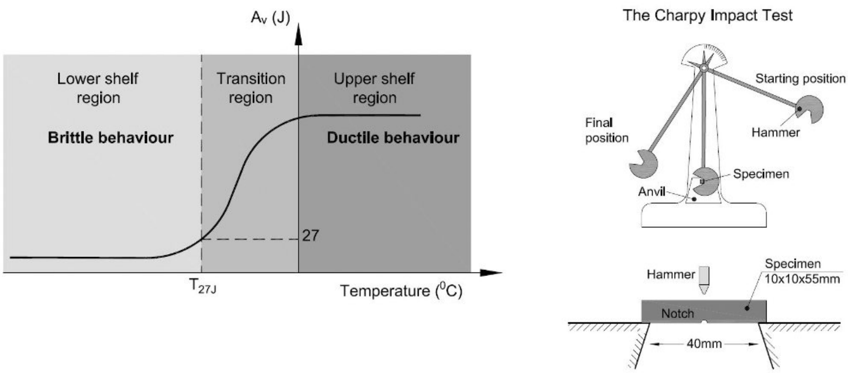


Fig. 1.6. Material toughness as a function of temperature

penetrates and the diameter of indentation measured. The Brinell Hardness (HB) is calculated as the ratio between the applied force and the indentation area. Although hardness exceptionally concerns structural applications, this test is mentioned here because it is a quick inexpensive test that allows an approximate determination of the tensile strength through the following expression:

$$f_u \text{ [MPa]} = 3.5 \text{ HB (Brinell Hardness)} \tag{1.28}$$

1.5.2.4 Z-properties

In order to avoid lamellar tearing, i.e. cracking perpendicular to the plate thickness, that is mainly due to non-metallic sulfur inclusions such as MnS, steel especially thick one has to be supplied with through-thickness properties when stressed in welded areas perpendicular to its thickness. These are also called Z-properties due to the fact that the x- and y-axes of a sheet correspond to the in-plane dimensions, while the z-axis to the thickness direction. The Z-properties are determined by a special tensile test in accordance to EN 10164 [1.45] that covers thicknesses between 15 and 400 mm. Samples of special geometry, not like the usual tension coupons, are taken over the thickness of the steel (Fig 1.7) and tested to tension. The Z-value is defined as the reduction of area according to eq. (1.29):

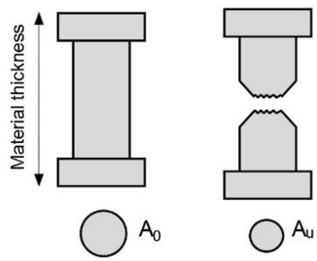


Fig. 1.7. Specimen geometry before and after the *Z*-test

$$Z = \frac{A_0 - A_u}{A_0} \cdot 100 \tag{1.29}$$

where:

A_0 is the cross-section area of the original specimen and
A_u the minimum cross-section area after fracture.

Three quality classes Z15, Z25, Z35 are distinguished with minimum Z-values 15%, 25% and 35% correspondingly.

1.5.3 Microstructure of steel

Steel is an alloy of a soft, ductile metal, iron (Fe), and a strong, brittle mineral, carbon (C). Steel is by definition the iron alloy that has C-content less than 2% by weight. With increasing C-content it becomes stronger, but more brittle and less workable. The microstructure of steel may be best understood on the iron-carbon phase diagram, in which the horizontal axis represents the C-content of the steel and the vertical axis the temperature during manufacturing. The C-content of structural steel is between 0.1% and 0.25% in order to control the toughness so that the phase diagram will be concentrated on such carbon contents as shown in Fig. 1.8.

Pure iron crystals are either body centered (bcc) or face centered (fcc), with 9 atoms or correspondingly 14 atoms. Iron composed exclusively of fcc crystals is knows as γ-iron or austenite that can dissolve around 2% C, while if composed of bcc crystals α-iron or ferrite that can dissolve less, around 0.02%, carbon. The first steel that forms when cooling the melt is austenite that can dissolve all carbon. By further cooling, some fcc crystal form so that for temperatures below the transition A_3-line both austenitic and ferritic forms exist. However, at temperatures below 723^0C the austenite decomposes into perlite and ferrite. Pearlite is steel with carbon content of

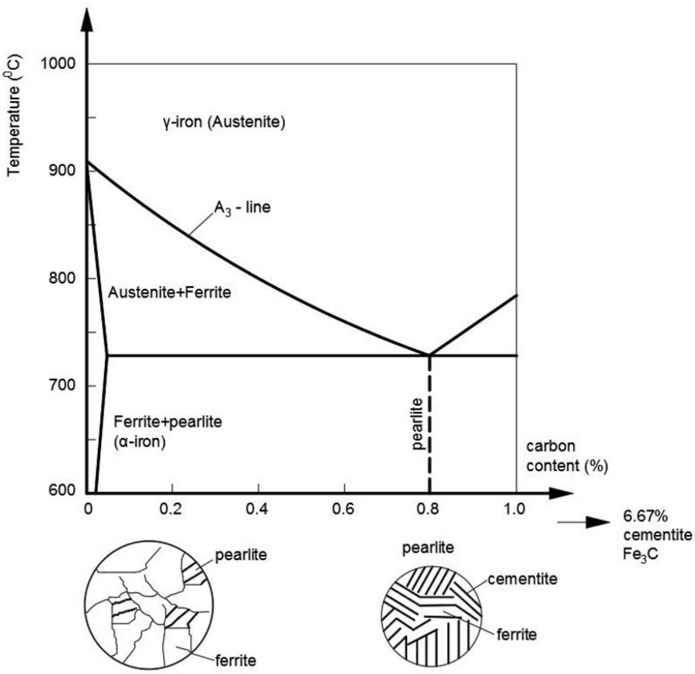


Fig. 1.8. Iron-carbon phase diagram

0.8% and is composed of ferrite and lamellas of iron carbides Fe_3C, called cementite. The former is soft and ductile, the latter, cementite, hard and brittle. The relative proportions of ferrite and pearlite adjust themselves to maintain the carbon content of the specific steel.

The transformations described in Fig. 1.8 occur when steel is allowed to cool in the furnace at low rates. Such steels are called fully annealed steels. When steel is cooling faster in the air, the proportion of pearlite increases slightly, but more important, the grain size of ferrite gets smaller and the pearlite lamellae finer resulting in an increase in strength, ductility and toughness. Air cooled steels are called normalized steels. However, when sprayed with cold water the cooling rate is very fast so that no ferrite and pearlite is formed, but another crystal form named martensite, while less rapid cooling rates lead to another form called bainite. Although the two last forms have higher strength and hardness, they are brittle, too hard and susceptible to cracking and should avoided. However, if the content in C or other alloys, more specifically the carbon equivalent (CE) is low, susceptibility to martensite formation and to low toughness decreases.

Solidification of the liquid metal starts from certain nuclei, the number of which determines the grain size. The more the nuclei, the smaller the grain size. Grain size is of main interest, since it largely influences the mechanical properties of steel, like strength and toughness. Smaller grains result in increase in strength without decrease of toughness. The relationship between yield strength and grain size is described by the Hall-Petch equation:

$$f_y = \sigma_0 + \frac{k}{\sqrt{d}} \; [\text{N/mm}^2] \tag{1.30}$$

where:

σ_0 [N/mm^2, or MPa] is constant depending on the composition of steel
k a constant, usually equal to 20 N/mm$^{-3/2}$
d is the grain diameter in [mm]

The grain size is influenced by:

- Alloying that increase the number of nuclei and therefore the number of grains. and
- The cooling rate. The grain diameter becomes smaller at accelerated cooling. Contrary, the grain size increases by reheating at very large temperatures or for very long time.

1.5.4 Making of steel and steel products

For the reasons explained before, a structural Engineer should have some basic knowledge on how steel is produced and with what processes the anticipated properties are achieved, taking into account that during fabrication and erection this steel may be welded and exposed to environmental or other loading conditions, e.g. to cyclic loading in the elastic or inelastic range, such for structures in seismic areas.

The routes to making steel and steel products are illustrated in Table 1.6. Route 1 is the blast furnace using iron ores as raw material. The iron ores are mixed with coke and lime to form a porous mixture called sinter. The blast furnace is fed at the top with sinter and coke, while compressed hot air is blast from holes of the furnace walls. The oxygen of the ascending air reduces the iron oxides, so that the melted material may be tapped at the bottom of the furnace, separately from the slag because it is heavier. The hot metal produced in the furnace is called pig iron and contains more than 4% carbon, as well as other impurities such as manganese (Mn), silicon (Si), sulphur (S) or phosphorus (P). Through the application of electric filters to clean the exhaust gases, preheating of the blasted air by the heat of the exhaust gases and recycling of the waste, the impact on the environment reduces to a minimum.

Pig iron from the blast furnace is further refined in the basic oxygen converter to produce steel, where oxygen is blown by a lance from the top of the converter in order to burn carbon and lower the C-content. The process is exothermic so that cold scrap is also added to lower temperature of the melt. Other elements such as Si, Mn, P are also burned and react with calcium and carbon oxides to build the slag.

In route 2 the raw material is only scrap which is charged in an electric arc furnace. The scrap is melted by an electric arc that is generated between the metal and the tips of three graphite electrodes. The developing temperature is very high, around 3500 °C, and is regulated by the voltage, the intensity of the current and the arc length. By burning the metal, a first slag is built that burns P and Si and lowers the C-content. After removing this slag, a second basic slag is built that reduces the S and O_2 content. The steel produced by this method is of low carbon content and high quality and is called electric steel.

Steel of even higher quality is produced after a secondary process in ladle furnaces (route 3). In these furnaces the material is homogenized, oxides are removed and gases such as S_2, or N_2 are reduced to very low levels. In addition, O_2 that was blast in the basic oxygen converter has to be reduced since steel with too high oxygen content is susceptible to breaking when red heated and accordingly useless. Oxygen is lowered in the blast furnace by addition of deoxidation agents, such as Mn, Si or Al, that bind oxygen to form oxides. These oxides are removed during the secondary process in the ladle furnaces by application of several methods such as stirring by injected Argon.

Subsequent steel is cast to solidify, either in a continuous or a discontinuous process. In the discontinuous process steel is cast in ingot moulds (route 5). During solidification, gas bubbles segregate due to the fact that gases are less soluble in the solid than in the melt. This leads for steels with low amount of deoxidation agents to "boiling" of the melt and ascending of gases to the top surface. This steel is called rimmed steel (FN), a name that was given from the fact that the ingot has an outside skeleton from pure iron and a central part where all impurities such as C, P, or S, concentrate. This segregation and the porosity of the material take place because blow holes remain in the mass during the rapid escape of the gases make the composition non-uniform and this steel not suitable for hot-working applications. On the contrary, if sufficient amount of deoxidation agents was added in the ladle furnace "fully killed" (FF) steels are produced that solidify quietly, are more homogenous but leave due to shrinkage a large crater at the ingot top that has to be subsequently cropped. This is uneconomical since all cropped material has to be put in the furnace again. Structural steels are usually deoxidized with Si alone and are called "semi-killed" or "balanced" steels that develop much smaller crater but have some small blow holes that close during rolling.

However, the most common and most economic casting process is route 4 of continuous casting, which is based on the oscillating movement of a water-cooled mould that forms a solid shell around the molten metal. After departure from the mould the strand is further cooled by water sprays and led by rolls until it is completely solid, where it is cut to length.

The final products are produced by hot-rolling. The casting material is reheated and passing through series of rolls in pairs moving in both directions to allow multiple passing, which give it progressively its final form. The material gradually cools down but temperatures do not fall during rolling below 900 °C in order to allow shaping without application of too high forces and damage of rolls. This is the conventional type of rolling as illustrated in Fig. 1.9a.

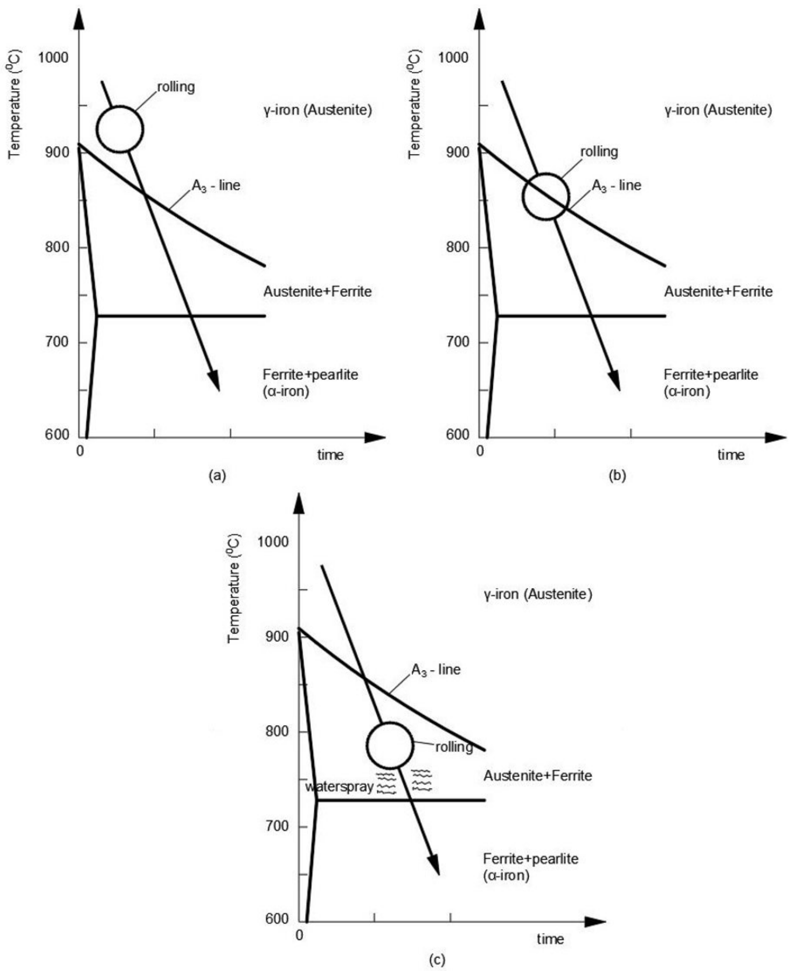
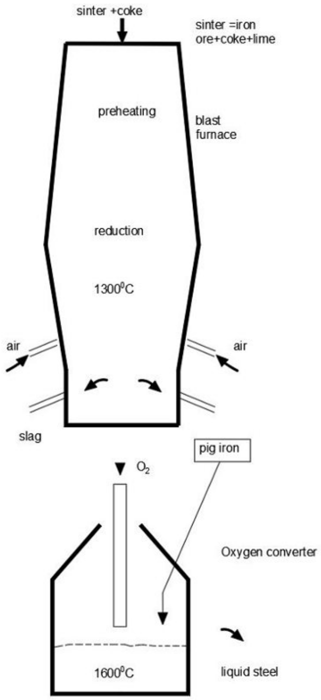
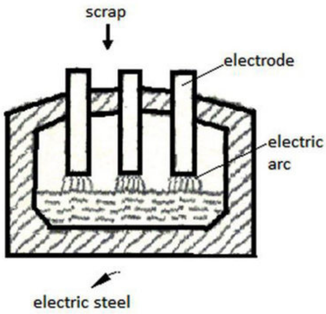


Fig. 1.9. a) Conventional rolling, b) normalizing rolling, c) thermomechanical rolling with quenching and self-tempering

Table 1.6. Routes to steel products

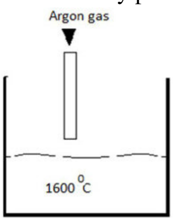


Route 1



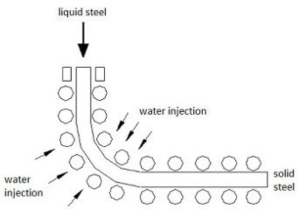
Route 2

Secondary processing



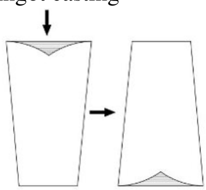
Route 3

Continuous casting



Route 4

Ingot casting



Route 5

Hot rolling

Long products (beams, bars, rods)

Flat products (coiled sheet, plates)

However, as discussed before, grain size plays a vital role for the mechanical properties of steel and this may be influenced by alloying and heat treatment. In previous times, heat treatment occurred after hot rolling, while today this happens during the rolling operation. The first rolling of this type is normalizing rolling, where steel is rolled at temperatures around the austenite-ferrite A_3 transition line (Fig. 1.9b). The other type is thermomechanical rolling, where steel is rolled at even lower temperatures, below the ferrite recrystallization temperature (Fig. 1.9c). This results in the formation of a fine grain steel with very high strength and toughness with less addition of alloy elements and therefore of better weldability. By spraying the steel surface during the last rolling pass with cold water, a process that is called accelerated cooling, the skin is quenched. Quenched steel has fine grain by low toughness. This is why quenching is accompanied by tempering, i.e. by reheating at temperatures around 600 °C so that toughness is restored. Steels produced by this process are called QT steels (from quenching and tempering). In thick products tempering occurs without reheating. Spraying with water stops before cooling of the core and the quenched skin is tempered by the heat released from the core during the cooling phase. This process is called quenching and self-tempering.

It is observed that in order to achieve a certain strength, conventionally rolled steels demand the highest alloy content, while heat treated, and especially QT, steels the least alloy content. Usual alloy may be accompanied by further fine alloying in small quantities with elements such as Niob (Nb) or Vanadium (Va) which leads to the formation of small particles that increase strength and lead to steels that resist the tendency to increase grain size during welding [1.46].

1.5.5 Structural steel grades

The designation system for structural steel is specified in the European Standard EN 10027 [1.47], while structural steel grades in EN 10025 [1.48]. As shown in Table 1.7, structural steel of a specific grade is specified by the letter S, meaning steel, a number expressing the nominal yield strength at small thickness, a symbol that gives the material toughness in longitudinal direction – rolling direction- at a given temperature and one or more additional symbols that indicate its treatment and /or its Z-properties. Low toughness steels JR and JO are rimmed FN-steels, while high toughness J2, K2 steels are fully killed FF-steels.

Table 1.7. Designation of steel grades according to EN 10027 [1.47]

Letter	*Number*	*Symbol 1*	*Symbol 2 (optional)*
S	Nominal yield strength [MPa]	(Table 1.8)	Treatment conditions (Table 1.9) and/or Z-properties (Table 1.10)

Mechanical properties of the most common structural steels to EN 10025 [1.48] as proposed in EN 1993-1-1 [1.13] are given in Table 1.11. These values are different from those specified in [1.48], due to the fact that Eurocode 3 makes simplifications

Table 1.8. Symbol 1 of Table 1.7. Minimum material toughness vs temperature

EN 10025 [1.48]	*Symbol 1*	*Temperature T* [°C]	*Minimum Charpy V-notch impact energy* [J]
Part 2	JR	20	27
Non-alloy structural steels	J0	0	27
	J2	−20	27
	K2	−20	40
Part 3			
Normalised / normalised rolled	N	−20	40
weldable fine grain structural steels	NL	−50	27
Part 4			
Thermomechanically rolled	M	−20	40
weldable fine grain structural steels	ML	−50	27

Table 1.9. Symbol 2 of Table 1.7. Examples of treatment condition

Symbol	+QT	+CR	+SR	G1	G2	G3	G4
Explanation	Quenched and tempered	Cold formed	Stress relieved	Rimmed steel	Killed steel	Delivery as agreed	Delivery according to producers choice

Table 1.10. Symbol 2 of Table 1.7. *Z*-properties

Symbol	+Z15	+Z25	+Z35
Explanation	Minimum reduction in area 15%	Minimum reduction in area 25%	Minimum reduction in area 35%

Table 1.11. Nominal values of yield and ultimate tensile strength for hot rolled structural steel according to [1.1], excerpts

Steel grate to	*Nominal thickness* $t \leqslant 40$ mm		40 mm < t ⩽ 80 mm		*Ultimate strain* ε_u
	f_y [MPa]	f_u [MPa]	f_y [MPa]	f_u [MPa]	
EN 10025-2					
S 235	235	360	215	360	26%
S 275	275	430	255	410	22%
S 355	355	510	335	470	22%
S450	440	550	410	550	19%
EN 10025-3					
S 275 N/NL	275	390	255	370	24%
S 355 N/NL	355	490	335	470	22%
S420 N/NL	420	520	390	520	19%
S460 N/NL	460	540	430	540	17%
EN 10025-4					
S 275 M/ML	275	370	255	360	24%
S 355 M/ML	355	470	335	450	22%
S420 M/ML	420	520	390	500	19%
S460 M/ML	460	540	430	530	17%
EN 10025-6					
S460Q/QL/QL1	460	570	440	550	

concerning the reduction of strength with increasing thickness. It should be mentioned that the designer could specify a steel with strength properties beyond those of EN 10025 or EN 1993-1-1, e.g. an S 355 with guaranteed yield strength 355 MPa at 120 mm thickness.

Table 1.12. Chemical composition of the melt for flat and long products [% by weight] according to EN 10025, excerpts

Specification	*Grade*	max C *for thickness t in mm*			max Mn	max Si	max P/S	max N
		$t \leqslant 16$	$16 < .t \leqslant 40$	$t > 40$				
	S 235JR	0.17	0.17	0.20	1.40	–	0.040	0.012
	S 235J0	0.17	0.17	0.17	1.40	–	0.035	0.012
	S 235J2	0.17	0.17	0.17	1.40	–	0.030	–
	S 275JR	0.21		0.22	1.50	–	0.040	0.012
EN 10025-2	S 275J0	0.18		0.18			0.035	0.012
	S 275J2	0,18		0.18			0.030	–
	S 355JR	0.24		0.24			0.045	0.012
	S 355J0	0.20		0.22	1.60	0.55	0.040	0.012
	S 355J2	0.20		0.22			0.035	–
	S 355K2	0.20		0.22			0.035	–
	S 355M	0.14			1.65		M	0.015
	S 355ML	0.14			1.65		P 0.035	0.015
EN 10025-4	S 420M	0.16			1.70	0.5	S 0.030	0.025
	S 420ML	0.16			1.70		ML:	0.025
	S 460M	0.16			1.70		P 0.030	0,025
	S 460ML	0.16			1.70		S 0.025	0,025

Further on, EN 10025 specifies the chemical composition of steel providing maximum contents of C or other elements as determined by analysis of the melt or the ready product. Excerpts are summarized in Tables 1.12 and 1.13. It may be seen that the limit values are more stringent for the melt than for the piece.

Table 1.13. Chemical composition of the piece for flat and long products [% by weight] according to EN 10025, excerpts

Specification	*Grade*	max *C for thickness t in* mm			max M*n*	max *Si*	max P/S	max *N*
		$t \leqslant 16$	$16 < .t \leqslant 40$	$t > 40$				
EN 10025-2	S 235JR	0.19	0.19	0.23	1.50	–	0.050	0.014
	S 235J0	0.19	0.19	0.19	1.50	–	0.045	0.014
	S 235J2	0.19	0.19	0.19	1.50	–	0.040	–
	S 275JR	0.24		0.25	1.60	–	0.050	0.014
	S 275J0	0.21		0.21			0.045	0.014
	S 275J2	0.21		0.21			0.040	–
	S 355JR	0.27		0.27	1.70	0.60	0.050	0.014
	S 355J0	0.23		0.24			0.045	0.014
	S 355J2	0.23		0.24			0.040	–
	S 355K2	0.23		0.24			0.040	–
EN 10025-4	S 355M	0.14			1.65	0.5	M	0.015
	S 355ML	0.14			1.65		P 0.035	0.015
	S 420M	0.16			1.70		S 0.030	0.025
	S 420ML	0.16			1.70		ML:	0.025
	S 460M	0.16			1.70		P 0.030	0.025
	S 460ML	0.16			1.70		S 0.025	0.025

Other properties of structural steel are as following:

Modulus of elasticity: $E = 210000$ MPa
Poisson ratio: $\nu = 0.3$
Shear modulus: $G = \frac{E}{2\cdot(1+\nu)} = 81000$ MPa
Specific weight: $\gamma = 78.5$ kN/m^3
Coefficient of thermal expansion α_t [per °C] $= 12 \cdot 10^{-6}$

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Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosions resistance.
Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.

2
Models and methods of analysis

Abstract. Structural analysis in nowadays performed electronically with use of appropriate software. This chapter gives criteria to set up numerical models for the entire structure or parts of it. It presents analysis models for the most usual applications, such as single story industrial buildings, but also for other common types of steel structures. It gives methods to incorporate composite slabs and treat composite beams in multi-story buildings and shows how to use sub-models for parts of the structure and more elaborated models for structural details that need special investigation. It then presents methods of analysis including linear and non-linear methods in terms of non-linear material behavior and geometric non-linear behavior, possibly accounting for geometric, structural or equivalent geometric imperfections. The implications of different types of analysis are illustrated for simple structural systems. It outlines the Eurocode provisions concerning the cross-section classification that allows or not the application of plastic analysis and design for steel structures. It finally presents the types and values of geometrical imperfections provided by the Eurocode and the analysis methods prescribed by the Code as well as alternative proposals of the authors on their selection.

2.1 Introduction

Structural analysis is required in order to determine the structural response to loading and other actions in terms of internal forces and moments, stresses, deformations, strains or vibrations. This may be done by setting up appropriate numerical models that represent the real structure or parts of it. A global analysis model should be based on the following criteria:

- It should reflect the geometric and mechanical properties of the structural elements in terms of mass, stiffness and strength.
- It should correctly represent the behavior of connections between elements, taking into account their detailing and any eccentricities.
- It should include as many as possible structural elements of the main and secondary structural system and, possibly, of the cladding if it participates to structural stability.
- It should include the foundations and soil, if necessary.
- Loads and load combinations should be easily introduced.

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- It should be easily implemented.
- The resulting output should be such that enables easily the execution of the Code prescribed verifications.

Depending on the application, sub-models or local models may be used besides the global model to verify specific parts of the structure or study local effects for certain construction details.

It is today's state of the art to use finite element methods (FEM) in order to represent the structure and perform the analysis at the design. The global model is usually a 3D (three dimensional) FEM model composed of beam, truss or cable elements. 2D (two dimensional) sub-models or shell element models for local analysis and constructional details may be additionally used to verify specific elements or parts of the structure or to study local effects for certain construction details. Hand calculations based on simple structural systems may precede in the preliminary design phase to estimate geometric properties, sizes of elements, weights etc. In the following some of the most usual FEM models for steel and composite buildings will be presented.

2.2 Models for steel buildings and other types of steel structures

Single story industrial buildings constitute the most common application for steel structures worldwide. The complete structure may be subdivided into three parts: the primary, or main, system, the secondary system and the cladding for roof and walls. A typical example for such a building, along with its three-dimensional (3D) FEM model, is illustrated in Fig. 2.1. The main system is formed by repetitive plane trussed frames composed of columns and lattice girders. Secondary members are the purlins on the roof, the internal columns at the gable frames and the rails on the walls. Out-of-plane stability is ensured by bracing systems placed in the roof and along the longitudinal walls. The building skin is composed of roof and wall cladding elements.

External loads, such as snow or wind are transferred from the cladding elements to the secondary and the primary structure. In that sense the elements, whether cladding, or parts of the primary or secondary system, have a structural function in resisting and transferring loads from the point of their application to the foundation soil. However, as shown in Fig. 2.1, it is common practice to create a numerical 3D global analysis model that includes the elements of the main and the secondary system which are represented by beam type finite elements. Cladding elements are most frequently not included in the model for a number of reasons. First, they would be represented in the numerical model by shell elements which not all designers are familiar with their use, second because the in-plane stressed-skin behavior is usually

Fig. 2.1. Model for a single story industrial building

neglected in design since it is to a large extend dependent on the connection behavior between the panels and the substructure, then, if stressed-skip design would be accounted for a panel removal would make a new analysis necessary. However, the contribution of cladding elements is considered, if not in global analysis, in design when checking the lateral stability of adjacent members.

The presented 3D numerical model complies with the criteria set up in the introduction for global analysis models, allows the easy and transparent implementation of loads, defined automatically the self-weight of the structure, except cladding, and may be connected to appropriate software for fabrication of the steel structure. Structural elements are represented generally by beam elements or by truss elements when they transfer only axial forces. Specific types of truss elements are tension-only elements that cannot sustain compression. Physical elements so represented are for example slender diagonals of X-bracing systems.

Conventional beam elements have six (6) degrees of freedom (DOF) that correspond to three displacements (u_x, u_y, u_z) and three rotations $(\theta_x, \theta_y, \theta_z)$ per node. They are based on Euler-Bernoulli or Timoshenko beam theories that maintain the assumption that cross-sections remain plane after deformation. According to the Euler-Bernoulli theory cross-sections are perpendicular to the bending line, thus neglecting shear deformations, while according to Timoshenko theory rotation is allowed between the cross-section and the bending line so that shear deformations are included. The analysis results, when using such beam elements are the nodal displacements and rotations which represent the element DOFs, as well as the internal axial and shear forces (N, V_y, V_z) and the internal torsion and bending moments (M_x, M_y, M_z).

For members in which the effects of torsion, including lateral torsional buckling (LTB), are relevant, the seven (7) DOF beam element may be used. This element has the first derivative of either the total angle of twist (θ_x') [2.1], [2.2], [2.3] or its primary part [2.4] as the additional, 7$^{\text{th}}$, DOF. When this element is used, the analysis results include additionally the warping bimoment B_w and the primary and secondary torsion moments M_{xp} and M_{xs}. Besides the additional internal moments, these types of elements are able to include in linear buckling analysis LTB instability modes for single beam elements while 6 DOF beam elements are not. More sophisticated beam elements with even more, up to even 10, degrees of freedom, that account for special effects like shear lag, distortion etc. have been developed [2.5], [2.6], [2.7] and may be applied to more complicated conditions such as for bridge analysis.

A physical beam may be represented by a series of beam elements in a row, thus providing the distribution of deflections and of internal forces and moments along it. The type of connection with other elements is described by providing appropriate boundary conditions at its end nodes. Accordingly, for a simple supported beam zero rotations are provided at the corresponding nodes of the first and last beam element representing the physical beam. If a physical beam is represented by one beam element only, the boundary conditions are set at the nodes of this element.

Truss elements have one (1) DOF, the u_x-displacement, and are used for elements subjected to compression-tension such as pin ended diagonals of bracing members or trusses. Physical bars must be represented by one truss element and not more elements in a row, since there is no continuity between truss elements and the system becomes unstable. Truss elements cannot resist transverse forces and do not develop bending moments. Accordingly, if a physical beam is represented by a truss element, it is not possible to determine its stress condition due to transverse loads or its own weight since such loads are automatically transferred to the end nodes.

Special types of truss elements are tension-only elements. They are useful to represent physical elements, such as X-braces, in which compression forces are ignored in design.

Cable elements are specific type of tension-only elements that are associated with geometrical non-linear analysis. Here again, one rope or cable must be represented by one element only.

Connections between elements are idealized conventionally as hinged or rigid. Semi-rigid connections are idealized by means of translation and/or rotation springs.

Loads are introduced easily in the 3D-model by generation of load distribution areas, so it is possible to define free point, line and area loads. These loads are subsequently converted into equivalent beam loads acting on members, when the structure is idealized by means of beam elements. Self-weights of structural elements are accounted for automatically by the software. For this purpose, the z-axis in global analysis models shows usually down in vertical direction so that gravity loads receive positive values (Fig. 2.2).

3D models for global analysis of other types of steel structures are set up with similar considerations. Fig. 2.2 shows examples of global analysis models for pallet racking systems and telecommunication towers [2.8], [2.9]. It is mentioned that graphical outputs such as those of Fig.s 2.1 and 2.2 may be misleading. The representation of elements with the full cross-section may give the wrong impression that members are modeled by shell elements so that local effects like load introductions, eccentricities etc. are accounted for in analysis. However, when beam or truss elements are used, as usually in global analysis, structural members are represented by

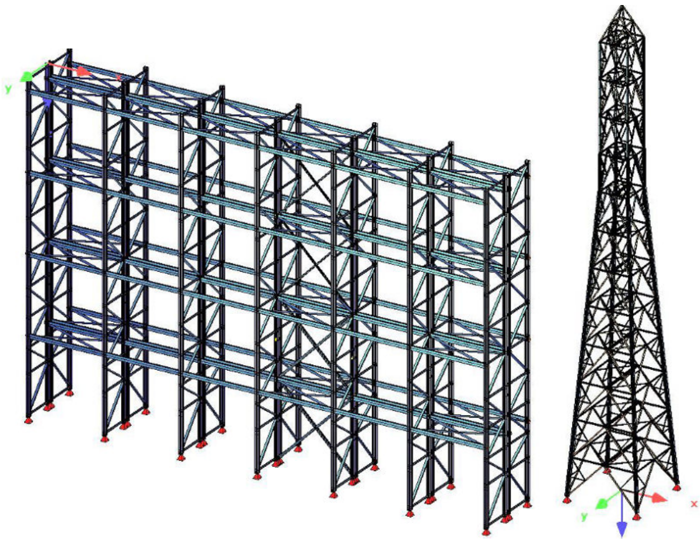


Fig. 2.2. Models for a pallet racking system and a telecommunication tower

a single line running through the centroid of the cross-section and analysis does not account for any local effects. Such effects should be examined separately, member eccentricities should be introduced by rigid links or kinematic dependencies between nodes etc. A complete global analysis model should include foundations and sometimes the soil in addition to superstructures. Foundations slabs are usually modeled by shell elements, foundation beams by beam elements, the soil is usually represented by Winkler type axial springs.

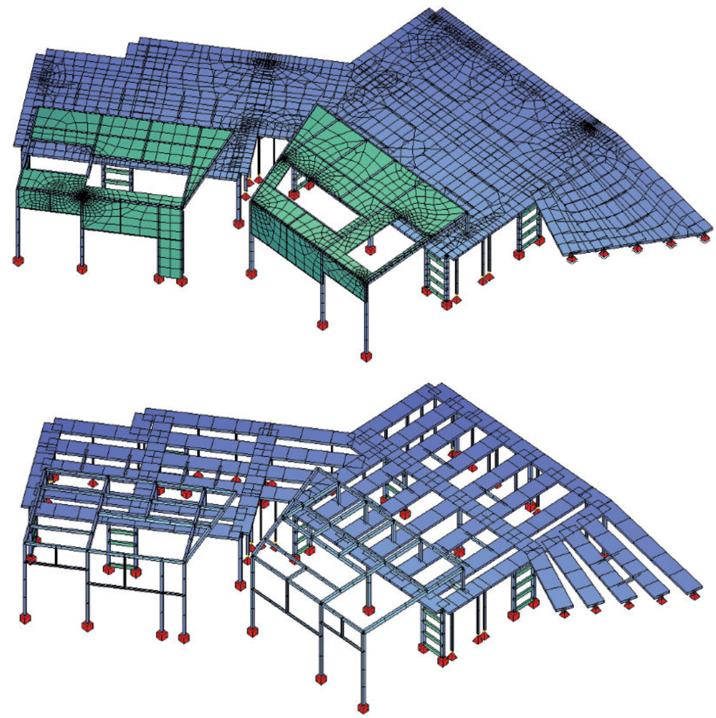
2.3 Models for composite buildings

Multi-story metal buildings for residential, office or similar use are most usually steel-concrete composite structures with differences to steel industrial buildings discussed before. One significant difference is that steel floor beams and profile steel decking act compositely with top concrete poured on-site to form composite floor systems. These composite systems have many benefits such as speed of construction, less cost, larger spans, or easy service integration. In addition, they offer diaphragm action being able to distribute horizontal loads to the columns which is of importance for buildings in seismic areas. The most advantageous modelling technique for composite floor systems is as following:

a) Floor beams are modelled by means of beam elements with composite cross-section composed of the steel beam and the cooperating concrete flange within its effective width. In cracked regions the concrete flange is missing in the cross-section, or alternatively the modulus of elasticity of concrete, E_c, is introduced with a very small value.
b) The composite decking is represented by means of shell elements. The axial and bending stiffness, EA and EI, of the shell elements is set to, practically, zero (0) in direction *parallel* to the floor beams. In direction *transverse* to the beams, EA and EI take the nominal values of un-cracked or cracked reinforced concrete. By this technique, the shell elements do not develop any axial forces or bending moments parallel to the composite beams, which accordingly resist the entire transverse loading.
c) The eccentric position of the deck is taken into account by placing the shell elements on top of the floor beam elements. Kinematic dependencies are set between the shell and beam elements to account for the shear connection between the slab and the steel beam.
d) In order not to consider the weight of the concrete twice, the concrete material of either the composite beam elements or the slab shell elements is considered weightless.

Fig. 2.3 shows analysis models for a single-story building with composite floor system [2.10]. Fig. 2.3a) illustrates the full model, where shell elements are used for the representation of both the composite decking and the sandwich panels used for roofing and the façade. The shell elements are not illustrated in Fig. 2.3b), so that the steel and composite beam elements for roofing and flooring are made visible.

In case that shell elements representing the deck are not included in the numerical model, the in-plane action of the deck may be taken into account by introduction of a



a)

b)

Fig. 2.3. Analysis model for a composite floor system showing a) the full model with shell elements representing the composite decking and b) only the floor beams with composite section composed of the steel beams and the concrete flange

diaphragm action externally. This is done by making all nodes of a floor kinematical dependent on a master node that is usually the mass center of the floor. Such models with imposed diaphragm action are usually softer than models where the deck is represented by shell elements.

Vertical concrete elements such as shear walls, cores of complex shape etc. should be preferably modelled by shell elements as illustrated in Fig. 2.4. The in-

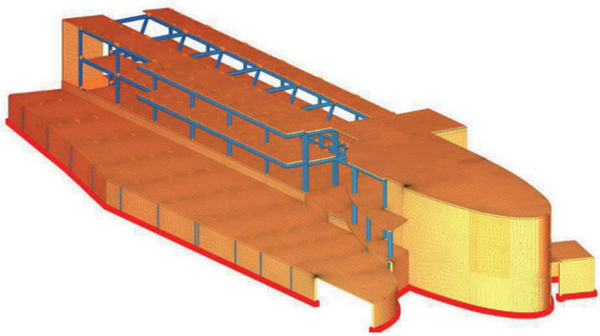


Fig. 2.4. Model of a building with representation of slabs and walls by shell elements

clusion of shear walls and cores in the global model by means of shell elements provides more accurate results in dynamic and stability analyses. Care must be given to account for the effects of cracking of concrete in analysis and design. The European seismic code EN 1998-1 [2.11] proposes to introduce 50% of the un-cracked bending and shear stiffness for all concrete elements in analysis. For composite steel-concrete beams cracking of concrete due to cyclic loading is taken into account in analysis by further reduction of the effective width of the concrete slab in accordance with [2.11].

2.4 Sub-models for structural parts or elements

Individual structural elements are sometimes designed on the basis of appropriate sub-models. Purlins, rails, floor beams may be for example modeled as continuous 2D beams, separately from the global structure as shown in Fig. 2.5a). Similarly, important parts of the structure may be modeled separately from the entire structure for optimization purposes and parametric analyses. As an example, a portal frame may be extracted from the complete structure and analyzed on a 2D sub-model as illustrated in Fig. 2.5b) [2.12]. Care must be given to the fact that 2D sub-models take into consideration only in-plane and neglect out-of-plane effects that have to be considered separately in design. Nevertheless, their application allows the implementation of parametric or refined non-linear analyses that might eventually lead to lighter sections.

a)

b)

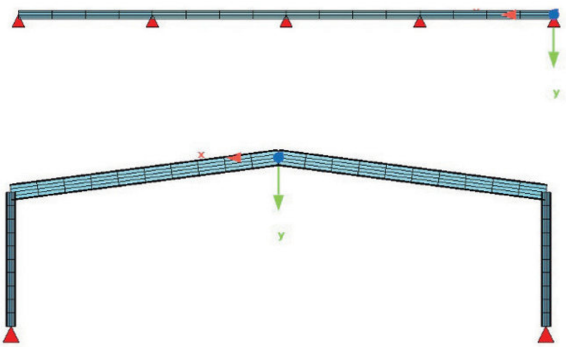


Fig. 2.5. 2D sub-models for a) purlins, rails, floor beams etc. represented as continuous beams and b) portal frames

2.5 Models for local analysis

For local analysis of specific construction details, more refined models may be employed. In such models various types of finite elements representing different structural parts may be combined. Fig. 2.5a) shows such an example of a model for the study of an anchorage detail. Steel plates are represented by shell elements, anchors by beam elements and the embedding concrete by non-linear springs. For a more detailed analysis of a short beam, the beam and its end plates are represented by 3D brick elements as shown in Fig. 2.5b).

Local analysis models may collaborate with the global models. Analysis is performed on the global model in order to determine global deformations and internal forces and moments. Subsequently the area of interest is separated and a more refined local model is built. The action effects as determined in the global model are introduced at the boundaries of the local model for a detailed study of that area. Such techniques were used for the determination of the crack initiation of welded beam-to-column joints subjected to cyclic seismic loading [2.13].

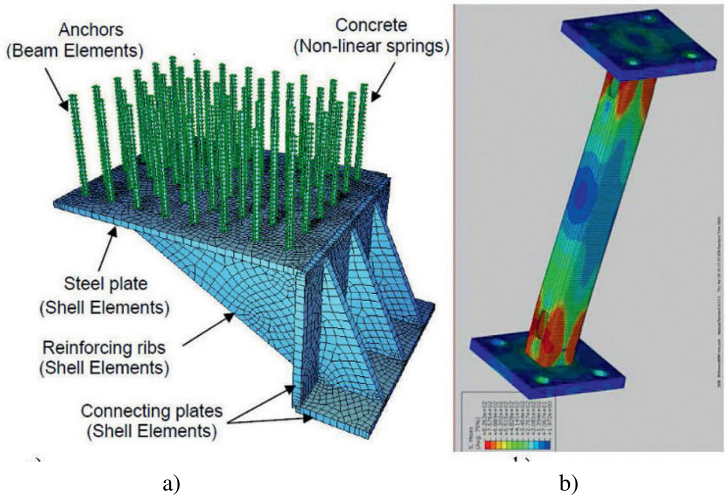


Fig. 2.6. Models a) for an anchorage detail and b) a short beam with end plates

2.6 Methods of analysis – General

The response of steel structures to loading is influenced by two non-linear effects, the non-linearity concerning material behavior and the geometric non-linearity. The first is due to large strains the second due to large displacements. Furthermore, real structures deviate from the ideal ones in respect to both geometric properties and stresses in the unloaded condition. This refers to both the components and the complete structure that deviate from the ideal one due to imperfections created during the manufacturing and erection process. The deviations of the real structure from the ideal geometry are called **geometric imperfections**. Additionally, due to various, mainly thermal, influences (welding, rolling, cutting) during manufacturing, members are restrained from deforming freely so that residual stresses are created that exist in members in the unloaded condition. The residual stresses constitute another type of imperfections called **structural imperfections**.

Methods of analysis are distinguished on whether and how they consider non-linear effects or imperfections. Table 2.1 presents the available methods of analysis following the nomenclature of EN 1993-1-6 [2.14].

Table 2.1. Methods of analysis for steel structures

		Inclusion of Imperfections	*Material behavior*	
			Linear	*Non-linear*
Geometric behavior	Linear	No	LA - LBA	MNA
	Non-linear	No	GNA	GMNA
		Yes	GNIA	GMNIA

LA: Linear Analysis
LBA: Linear Buckling Analysis
MNA: Materially non-linear Analysis
GNA: Geometrically non-linear Analysis
GMNA: Geometrically and Materially non-linear Analysis
GNIA: Geometrically non-linear Analysis with Imperfections
GMNIA:Geometrically and Materially non-linear Analysis with Imperfections

The main characteristics of the various methods are presented as follows. Further details are given in the relevant sections.

2.6.1 Linear analysis (LA)

Displacements and strains are small so that material behavior is elastic and analysis may be performed on the basis of the initial, un-deformed geometry of the structure. This analysis is also called elastic analysis according to 1^{st} order theory. Design is made separately by application of code-prescribed formulas.

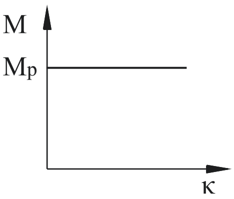
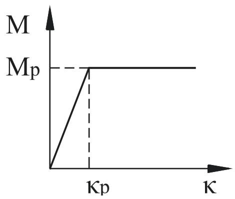
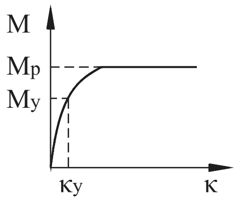
2.6.2 Linear buckling analysis (LBA)

This analysis provides buckling eigenvalues and buckling modes under the assumption of small displacements, elastic material behavior and no imperfections. It is a useful tool to explore to what extend geometric effects influence the structural behavior and to provide deformed structural shapes that may be used as geometrical imperfections.

2.6.3 Materially non-linear analysis (MNA)

Displacements are small but strains are large. Analysis may be performed on the basis of the initial, un-deformed geometry of the structure but the effects of non-elastic irreversible strains must be taken into account. This analysis is also called 1^{st} order plastic analysis. For frame structures it is further distinguished into rigid plastic analysis, plastic hinge analysis or plastic zone analysis as illustrated in Table 2.2. It may be used for design as long as geometric effects and imperfections can be ignored.

Table 2.2. Non-linear moment-curvature diagrams of cross-sections for beam elements

	Cross-section behavior		
Moment – curvature diagram			
Types of frame analysis	Rigid plastic analysis	Plastic hinge analysis	Plastic zone analysis

2.6.4 Geometrically non-linear elastic analysis (GNA)

Due to large displacements, equilibrium is defined in the deformed state of the structure under loading. Material behavior is elastic. This analysis is also called elastic analysis according to 2nd order theory. It is appropriate for stability investigations up to the buckling load under moderate displacements. Rotations are limited to approximate 350 mrad and are considered small so that the kinematic relation for the curvature may be linearized to write:

$$\kappa = -w'' \tag{2.1}$$

where (Table 2.3):

κ is the curvature and
w the deflection

For large displacements and even larger rotations, e.g. for cables, 3rd order or large displacements theory applies which is able to follow the structural response deep into the post-buckling region. Its main difference to 2nd order theory is the kinematic relation for the curvature that is non-linear according to:

$$\kappa = -\frac{w''}{\sqrt{(1+w'^2)^2}} \tag{2.2}$$

Analyses according to 2nd and 3rd order theories are also called linear and non-linear stability analyses. 3rd order, or large displacement, analysis is more relevant to plated or shell structures and hardly finds application in usual steel structures. This is due to the fact that very large displacements do general not develop in common civil engineering structures due to limitations imposed by the serviceability limit state (SLS).

2.6.5 Geometrically and materially non-linear analysis (GMNA)

This analysis combines MNA and GNA analyses and is also called 2nd order plastic analysis which might be rigid plastic, plastic hinge or plastic zone analysis for framed structures.

2.6.6 Geometrically non-linear elastic analysis with imperfections (GNIA)

This is a GNA analysis that considers initial imperfections. For simplicity, analysis is usually made with equivalent geometrical imperfections in which the influence of structural imperfections is accounted for in the geometrical imperfections. The shape and value of imperfections depend on the structure and element considered and are usually provided by the Codes. Imperfection may follow the shape of the fundamental buckling mode with an appropriate scale. Design is made separately, but basically on cross-section level since imperfections and geometric effects have already been taken into account.

2.6.7 Geometrically and materially non-linear analysis with imperfections (GMNIA)

This analysis takes into consideration all relevant non-linear effects and imperfections. It is therefore appropriate for both analysis and design and helps finding the true limit load.

2.7 Linear analysis (LA)

Linear analysis is the most usual one in practical applications and explicitly used for verifications at the serviceability limit state. It is simpler, quicker, more straightforward and has the advantage that it allows linear superposition of the results of the individual load cases for combined effects. However, it does not consider the effects of stability and plasticity that must be accounted for later in design.

Structural elements for framed structures are represented by a line going through the cross-sections centroid. According to Bernoulli, cross-sections remain plane and are perpendicular to the deformed element axis. Loads are conservative. Stresses and strains develop only along the beam axis x. Axes y and z of the element coincide with the major and minor principal axes. Three of the resulting internal forces and moments (N, M_y, M_z) refer to the gravity center G of the cross-section, while the other three (M_x, V_y, V_z) to the shear center S, Fig. 2.7. Axial stresses σ are not indexed but refer to the longitudinal axis x.

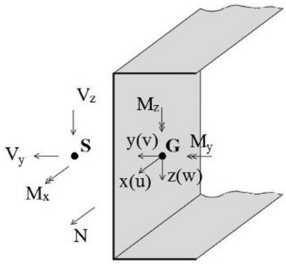
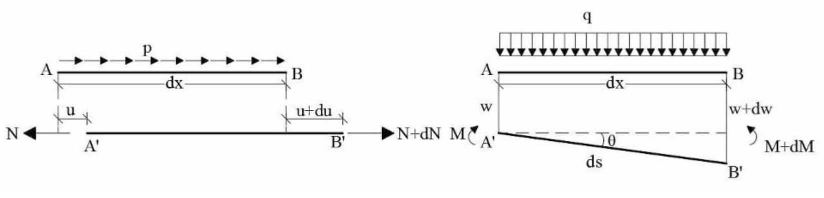


Fig. 2.7. Designations for axes, displacements and internal forces and moments of a 6 DOF beam element

The basic relations of the truss and beam element are summarized in Table 2.3. The left side of the total potential is the work done by the external loads, while the right side the elastic strain energy stored in the system.

Table 2.3. Basic relations for elastic truss and Euler beam element, 1st order theory

	Truss element	*Beam element*
		
Kinematics	$\varepsilon = \dfrac{ds - dx}{dx} = \frac{du}{dx} = u'$	$\theta = \dfrac{dw}{dx} = w'$
Elasticity	$N = E \cdot A \cdot \varepsilon = E \cdot A \cdot u'$	$M = -E \cdot I \cdot w''$
Equilibrium	$N' + p = 0$	$(E \cdot I \cdot w'')'' - q = 0$
Total potential V	$\int \left(\frac{1}{2} \cdot N \cdot \varepsilon - p \cdot u \right) \cdot dx =$ $\int \left(\frac{1}{2} \cdot E \cdot A \cdot u'^2 - p \cdot u \right) \cdot dx$	$\int \left(\frac{1}{2} \cdot M \cdot \kappa - q \cdot w \right) \cdot dx =$ $\int \left(\frac{1}{2} \cdot E \cdot I \cdot w''^2 - q \cdot w \right) \cdot dx$

In the Euler beam, displacements due to shear forces were neglected. If they are taken into account, the sliding angle appears as an independent variable and the deformed cross-section is not perpendicular to the axis of the beam but is inclined by this angle γ, Fig. 2.8. The shear force is no more parallel to the cross-section's axis but perpendicular to the beam axis. The beam element with consideration of shear deformations is called Timoshenko beam. The basic relations for the Timoshenko

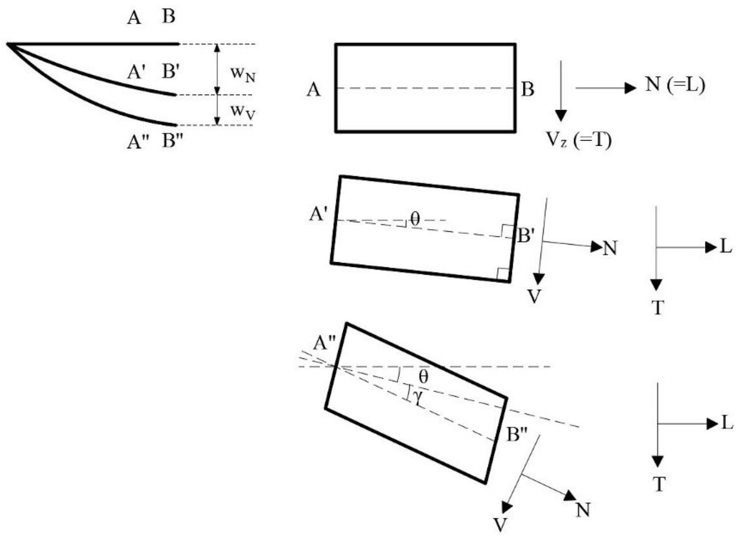


Fig. 2.8. Timoshenko beam element with shear deformations

beam are summarized in Table 2.4. For the Timoshenko beam a new stiffness term, the shear stiffness S_v, appears in addition to the axial stiffnes EA, and the bending stiffness EI.

Table 2.4. Basic relations for the Timoshenko beam, 1^{st} order theory

Kinematics	$w = w_M + w_V \quad w' = \theta + \gamma \quad \kappa = \theta'$
Elasticity	$M = -E \cdot I \cdot \theta' \quad V = s_v \cdot \gamma$
Equilibrium	$M'' = V' = -q \rightarrow (E \cdot I \cdot w'')'' + \left(\frac{EI}{S_v} \cdot q\right)'' - q = 0$
Total potential V	$\int \left(\frac{1}{2} \cdot M \cdot \kappa + \frac{1}{2} \cdot V \cdot \gamma - q \cdot w\right) \cdot dx =$ $\int \left(\frac{1}{2} \cdot E \cdot I \cdot w_M''^2 + \frac{1}{2} \cdot S_v \cdot w_V'^2 - q \cdot w\right) \cdot dx$

FEM beam elements account for the effects of shear flexibility, which is the inverse of the shear stiffness. Accordingly, shear deformations are always included in electronic FEM calculations. On the contrary, simple hand calculations mostly ignore the contribution of shear deformations. Caution is therefore needed in cases where shear deformations are important, such as for short span beams, concentrated forces, lattice girders or girders with web openings, due to the fact that either the shear forces are high or the shear stiffness low.

The shear stiffness for rolled beams or plated girders is equal to:

$$S_v = G \cdot A_v \tag{2.3}$$

where:

G is the shear modulus and
A_v is the shear area of the cross-section, to be defined in Section 3.5.

The shear stiffness of lattice girders is determined through application of the virtual work principle as shown in the following example. The lattice girder of Fig. 2.9 has

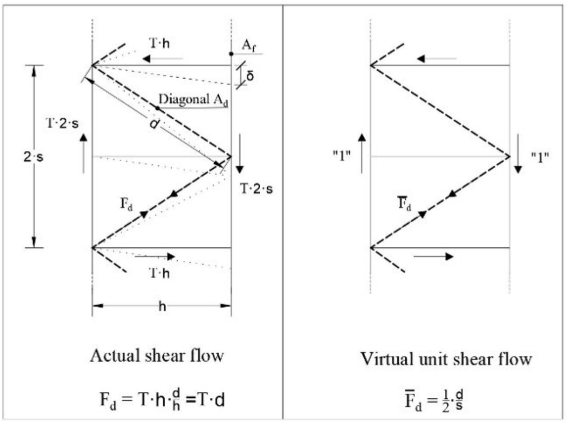


Fig. 2.9. Determination of shear stiffness for a lattice girder

the height h, a center-to-center distance between post-beams s, while its diagonals have a cross-section area $A_{d.}$ According to the principle of virtual work the displacement δ can be calculated from:

$$\text{“1”} \cdot \delta = \sum \frac{F_d \cdot \overline{F}_d}{E \cdot A_d} \cdot d_i = \frac{F_d \cdot \overline{F}_d}{E \cdot A_d} \cdot 2 \cdot d \rightarrow \delta = \frac{T \cdot d^3}{E \cdot A_d \cdot s} \tag{2.4}$$

The shear stiffness is then equal to:

$$S_v = \frac{T}{\gamma} = \frac{T}{\delta \cdot h} = \frac{E \cdot A_d \cdot s}{d^3 \cdot h} \tag{2.5}$$

The shear stiffness of other types of lattice girders and battened girders is given in Table 2.5. Forces in the diagonals and the post beams are also included.

2.8 Linear buckling analysis (LBA)

At the initial loading stages, a structure follows a stable load path, which is a straight line for a linear elastic perfect structure. Loading may be increased proportionally up to the point where the load path bifurcates. This load corresponds to the critical buckling load, also called Euler buckling load. Mathematically, this corresponds to the solution of an eigenvalue problem of the form:

$$(K + \alpha_{cr,i} \cdot K_G) \cdot \psi_i = 0 \qquad i = 1, \ldots n \tag{2.6}$$

where:

K is the stiffness matrix of the structure
K_G is the geometric stiffness matrix of the structure at the imposed design loads F_{Ed}
$\alpha_{cr,i}$ is the multiplier of the reference load to reach the buckling eigenvalue i and
ψ_i is the shape of the corresponding buckling mode i.

The buckling load for the i^{th} buckling mode is determined from:

$$F_{crit,i} = \alpha_{cr,i} \cdot F_{Ed} \tag{2.7}$$

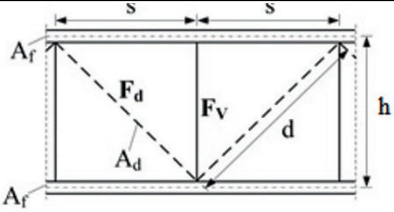
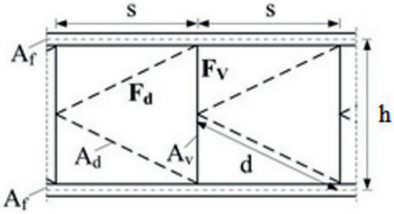
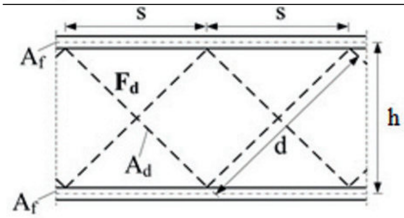
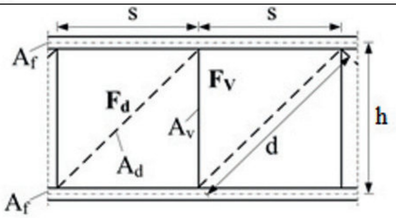
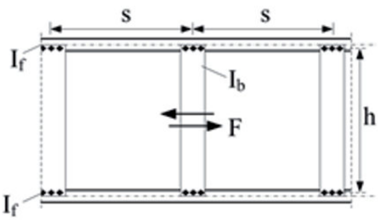
Buckling eigenvalues are found such that $\alpha_{cr,i} < \alpha_{cr,i+1}$. Obviously, the critical buckling load corresponds to the lowest eigenvalue $i = 1$.

Linear buckling analysis is associated with following assumptions:

- Linear elastic behavior
- Small deflections and
- Proportional loading, i.e. all imposed loads must be multiplied simultaneously by the scalar λ_i to reach the critical buckling load.

LBA is a useful tool for detecting the susceptibility of structures to instability and allows the determination of buckling loads and buckling mode shapes. Buckling loads are determined with their value. On the contrary, buckling modes may be found only by their shape but not by their absolute value. Nevertheless, buckling shapes are of

Table 2.5. Shear stiffness of lattice girders [2.15]

	$S_v = \dfrac{E \cdot A_d \cdot s \cdot h^2}{d^3}$ Forces: $F_v = 0,\ F_d = T \cdot d$
	$S_v = \dfrac{E \cdot A_d \cdot s \cdot h^2}{2 \cdot d^3}$ Forces: $F_v = T \cdot \frac{b}{2},\ F_d = T \cdot d$
	$S_v = \dfrac{E \cdot 2 \cdot A_d \cdot s \cdot h^2}{d^3}$ Forces: $F_d = T \cdot \frac{d}{2}$
	$S_v = \dfrac{E \cdot A_d \cdot s \cdot h^2}{d^3}$ Forces: $F_v = T \cdot b,\ F_d = T \cdot d$
	$S_v = \dfrac{24 \cdot E \cdot I_f}{s^2 \cdot \left(1 + \dfrac{2 \cdot I_f \cdot h}{I_b \cdot s}\right)}$ Forces: $F = T \cdot s$

Notation

A_d = cross-sectional area of diagonals
I_f = 2nd moment of area of flanges
I_b = 2nd moment of area of post beams
h = height of lattice girder
s = distance between post-beams
F_d = force in diagonals
F_v = force in post beams

importance due to the fact that if appropriately scaled they may be used in analysis as initial geometric imperfections.

LBA may be used to determine buckling length coefficients in frame analysis. Indeed, if the axial force of a column due to the design loads is equal to N_{Ed} the critical buckling load of this column at the buckling mode i is:

$$N_{crit,i} = \alpha_{cr,i} \cdot N_{Ed} \tag{2.8}$$

or according to Euler's formula

$$N_{crit,i} = \frac{\pi^2 \cdot E \cdot I}{L_{cr}^2} = \frac{\pi^2 \cdot E \cdot I}{(\beta L)^2} \tag{2.9}$$

The combination of (2.8) and (2.9) provides the buckling length coefficient of the columns for the relevant buckling mode, as following:

$$\beta = \frac{1}{L} \cdot \sqrt{\frac{\pi^2 \cdot E \cdot I}{(\beta L)^2}} \tag{2.10}$$

where:

$\beta = L_{cr}/L$
L is its system length
L_{cr} is its buckling length
$\alpha_{cr,i}$ is the eigenvalue of the relevant buckling mode
N_{Ed} is the design axial force in the column under consideration
EI is the bending stiffness of the column.

The critical point in this calculation is the correct selection of the appropriate buckling mode. This is simple for single story buildings, since this mode is the 1st sway mode. However, for multi-story buildings different modes may be critical for the columns of different stories as outlined in the next example.

Figure 2.10 shows the geometrical properties of a two-story three bay building. The beams are loaded by a uniform loading $p = 75$ kN/m. In order to imitate multi-story buildings, additional point loads V_1, $V_2 = 2 \times V_1$ are introduced at the nodes of the top story, while in addition horizontal loads H_1, H_2 are applied at the floors in order to account for sway imperfections.

Four cases of loading are taken into consideration, with sum of vertical concentrated forces to 0, 4500, 9000 and 9000 kN and corresponding sum of horizontal forces 15, 37.5, 37.5 and 50 kN. Linear buckling analysis was performed for each case providing the critical eigenvalues α_{cr} as well as the buckling length coefficients β of the columns. The shape of the 1st and 2nd buckling modes are illustrated in Fig. 2.10, indicating that the 1st buckling mode is a sway mode of the first floor, while the 2nd buckling mode a sway mode of the second floor. Table 2.6 shows the buckling length coefficients of the external and internal columns for the two floors calculated on the basis of the eigenvalues of the two modes $\alpha_{cr,1}$ or correspondingly $\alpha_{cr,2}$. Following observations may be made:

a) The buckling length coefficients for internal and external columns do not very much depend on the loading conditions, if they are calculated on the basis of

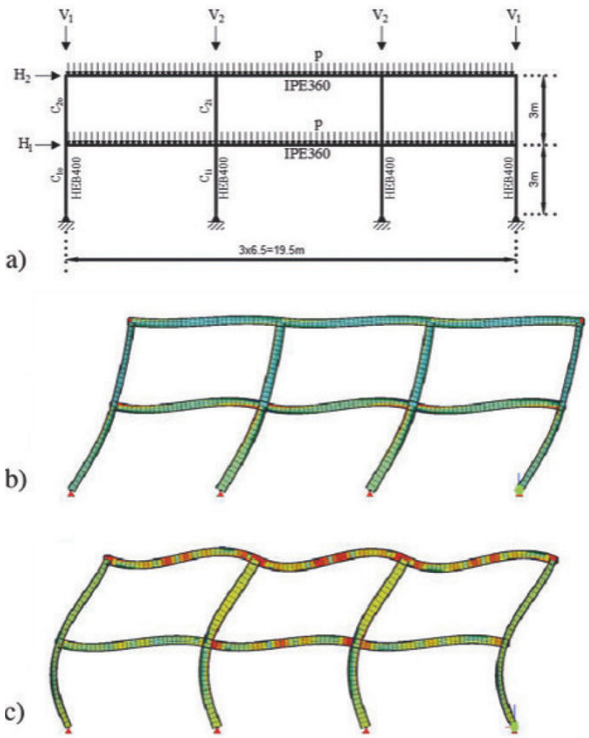
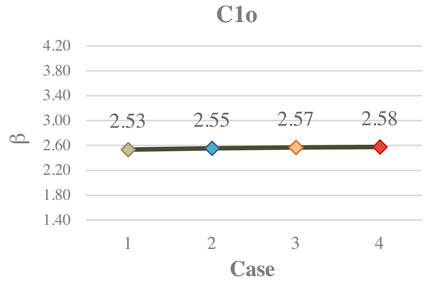


Fig. 2.10. a) Two-story building, b) 1st buckling mode, c) 2nd buckling mode

the correct buckling mode. Accordingly, the β-factors of the columns of the first floor, both external and internal, should be determined from the first buckling mode, while those of the columns of the second floor from the second buckling mode.

b) This may be additionally checked as following: the β-factors of the 2^{nd} floor calculated from the first buckling mode are larger than those of the 1^{st} floor. However, this should not be correct since the columns of the 2^{nd} floor are restrained at both ends from the beams while those of the 1^{st} floor are hinged at their bottom.
c) The buckling length coefficients of columns for a specific story are not all equal but depend on the degree of stressing, the most stressed ones having smaller buckling length coefficients than the less stressed ones. This is due to the fact that the more stressed columns would buckle first and are laterally supported by the less stressed ones until both buckle simultaneously at a certain sway mode. In this example most stressed are the external columns with the smaller profile despite the fact that they are subjected to lower axial loading.
d) Concluding for the example under consideration the β-factors of the first floor may be taken equal to 2.55 or 3.05 and of the second floor equal to 1.50 or 1.77 for the external and internal columns correspondingly.

Table 2.6. Buckling length coefficients $\beta = L_c/L$ for the columns of the two-story building of Fig. 2.10 for different values of the vertical loading

<table>
<tr><th>Buckling modes</th><th>factors β</th></tr>
<tr><td rowspan="4">First buckling mode</td><td>
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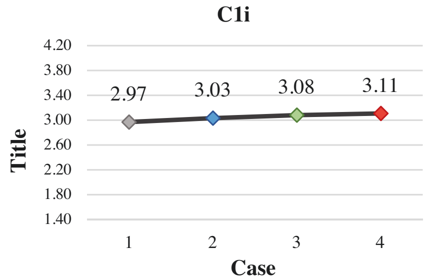
<tr><td>
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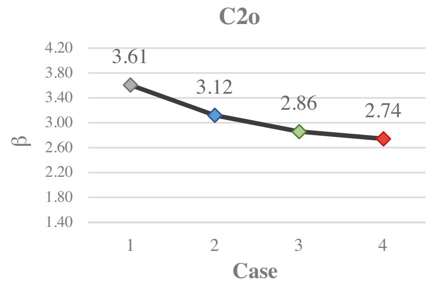
<tr><td>
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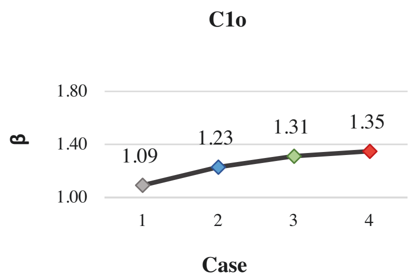
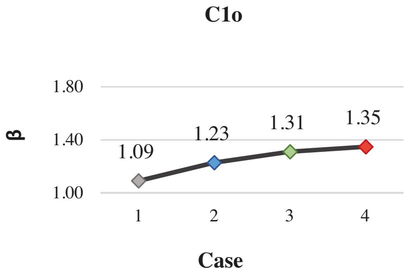
<tr><td>
</td></tr>
</table>

Table 2.6. *continued*

<table>
<tr><th>Buckling modes</th><th>factors β</th></tr>
<tr><td></td><td>
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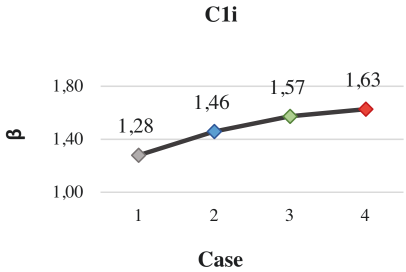
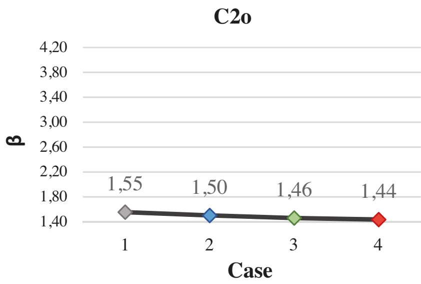
<tr><td></td><td>
</td></tr>
<tr><td>Second buckling mode</td><td>
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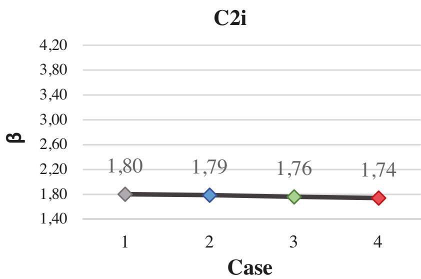
<tr><td></td><td>
</td></tr>
<tr><td>Remarks</td><td>Case 1: $\alpha_{cr,1} = 10.78$ Case 2: $\alpha_{cr,1} = 6.95$
Case 3: $\alpha_{cr,1} = 4.07$ Case 4: $\alpha_{cr,1} = 2.51$</td></tr>
</table>

In general, framed structures or structural elements are less susceptible to buckling phenomena and geometric effects if the multiplier of the lowest eigenvalue is higher than 10, see eq. (2.11), which roughly corresponds to the condition that the influence of 2^{nd} order effects is less than 10%.

$$\alpha_{cr,1} > 10 \tag{2.11}$$

It may be seen that the frame under consideration is susceptible to geometric effects for all cases but the first one.

2.9 Materially non-linear analysis (MNA)

2.9.1 Non-linear cross-section behavior

Steel does not behave elastically at all stress levels, but is subjected to yielding and strain hardening at larger strains so that its stress-strain curve becomes non-linear. The deformations in the non-linear range are no more reversible as in the linear one. The influence of strain hardening may be neglected by setting the tangent modulus $E_t = 0$, or accounted for by a small value of it, as for example $E_t = E/1000$. Fig. 2.11 shows bilinear approximations of the $\sigma - \varepsilon$ curve, applicable for carbon steels. High strength steels exhibit non-linear behavior over a wider range of deformations which is modeled by more elaborated curves.

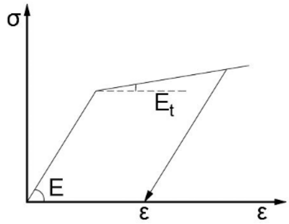


Fig. 2.11. Bilinear stress-strain curve for carbon steels

Non-linear material analyses in which the structure is represented by shell or fiber element models are based on the shape of the assumed $\sigma - \varepsilon$-curve. However, for frame analysis where beam elements are used in the model, or for hand calculations the cross-section's behavior as discussed subsequently becomes of importance.

Assuming a bi-linear material response without strain hardening, strains and stresses complying with the Bernoulli law, that sections remain plane, may be determined as shown for a doubly symmetric I cross-section in Fig. 2.12. The cross-section has an elastic core within the height z_e and yields outside this core.

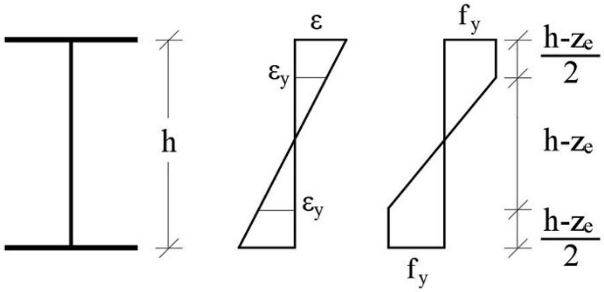


Fig. 2.12. Strain and stress distribution on a cross-section

The resulting moment is equal to:

$$M = 2 \cdot \left[\frac{A_f \cdot h}{4} + \frac{1}{2}(h - z_e) \cdot \frac{h + z_e}{4} \cdot t + \frac{1}{2} \cdot \frac{z_e}{2} \cdot t \cdot \frac{2}{3} \cdot \frac{z_e}{2} \right] \cdot f_y \tag{2.12}$$

and the curvature

$$\kappa = 2 \cdot \varepsilon_y / z_e \tag{2.13}$$

The corresponding yield moment M_{el} and yield curvature κ_{el} are determined from the above expressions by setting $z_e = h$. Moments and curvatures in dimensionless form are given by:

$$m = \frac{M}{M_{el}} \tag{2.14a}$$

$$\bar{\kappa} = \frac{\kappa}{\kappa_{el}} \tag{2.14b}$$

Their relation is expressed by:

- Elastic region $\bar{\kappa} \leqslant 1$

$$m = \bar{\kappa} \tag{2.15a}$$

- Plastic region $\bar{\kappa} > 1$

$$m = \frac{M}{M_{el}} = \frac{3 \cdot (1 + \alpha_f) - (1 - \alpha_f)/\bar{\kappa}^2}{2 \cdot (1 + 2 \cdot \alpha_f)} \tag{2.15b}$$

where:

A_f is the sum of the areas of the two flanges
A_w is the area of the web
$A = A_f + A_w$ is the total area of the cross-section
$\alpha_f = \frac{A_f}{A}$ is the ratio between the area of the flanges to the total area of the cross-section.

For a rectangular cross-section it is $\alpha_f = 0$, for a cross-section composed of two flanges $\alpha_f = 1$. IPE-sections have smaller α_f values than H-sections.

Fig. 2.13 shows curves resulting from numerical applications of equations (2.14) and (2.15). The moments at the largest value of m are the plastic moments of the cross-section M_{pl} and as shown in the Fig. 2.13 are dependent on the shape of the cross-section and accordingly the ratio α_{pl} which is the shape factor of the cross-section:

$$\alpha_{pl} = \frac{M_{pl}}{M_{el}} = \frac{3 \cdot (1 + \alpha_f)}{2 \cdot (1 + 2 \cdot \alpha_f)} \tag{2.16}$$

Fig. 2.13 shows that the shape factor of the rectangular cross-section is 1.50 and 1.0 for the cross-section composed of two flanges.

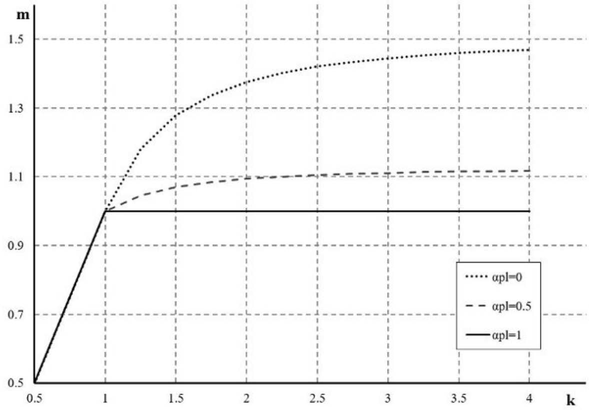


Fig. 2.13. Moment-curvature curves of cross-sections related to the corresponding values at first yield

Table 2.2 shows that the type of MNA frame analysis depends on the assumptions of the M-κ-curves and is:

- Rigid plastic analysis when elastic deformations are neglected. This analysis is appropriate for the determination of collapse loads by means of quick hand calculations.
- Plastic hinge analysis in which the cross-section is considered elastic until it reaches its plastic moment and then becomes a, plastic, hinge.
- Plastic zone analysis which takes into account gradual plasticity, both within the cross-section and along the beam. This is the most refined method, usually applicable in FE calculations with fiber beam models.

It should be said that the application of these methods requires that cross-sections have sufficient rotation capacity. They must be therefore class 1 as explained later in 2.9.3.

2.9.2 Collapse loads

Collapse loads without knowledge of intermediate stages may be directly determined by application of the theory of plasticity that suggests that the limit state is achieved when:

a) The equilibrium conditions are met.
b) For all cross-sections it is $|M| \leqslant M_p$.
c) The kinematic mechanism is achieved and
d) The plastic energy is positive.

Since it is generally difficult to meet all the above conditions, the static or kinematic theorems are used instead which neglect one of the above conditions and therefore provide approximate solutions. The static theorem neglects condition (c) and leads to low, conservative, values, whereas the kinematic one neglects condition (b) and leads to upper, non-conservative values.

The application of the two theorems in the beam of Fig. 2.14 with one clamped support on its left end and one simple support at its right end is shown in the following:

2.9.2.1 Static theorem

The moment diagram of Fig. 2.14a is statically allowed because it meets the equilibrium conditions and no moment exceeds the plastic moment. The collapse load is determined by equating an assumed value of the moment at span, which does not exceed the plastic moment, in this example taken equal to:

$$M_{span} = 0.75 M_p$$

to the moment at mid-span from statics:

$$M_{span} = \frac{q \cdot l^2}{8} - 0.5 \cdot M_p$$

Equating the two moments yields the collapse load:

$$q_{u,stat} = \frac{10 \cdot M_p}{l^2} \tag{2.17}$$

It should be said that for the support conditions considered, the maximum moment does not appear at mid-span but at a distance $5/8 \cdot l$ from the clamped end and is equal to $\frac{9}{128} \cdot q \cdot l^2$.

2.9.2.2 Kinematic theorem

A possible kinematic mechanism is shown in Fig. 2.14b. This mechanism violates condition (b) since the moments near the clamped end are higher than the plastic moment. The collapse load is determined by equating the two works:

Work of external forces: $\frac{q_u \cdot l}{2} \cdot \theta \cdot \frac{l}{2}$
Work of internal forces: $M_p \cdot \theta + M_p \cdot (2\theta)$

The collapse load is then equal to:

$$q_{u,kin} = \frac{12 \cdot M_p}{l^2} \tag{2.18}$$

which is higher than the load previously determined by the static theorem.

It should be mentioned that in statically determined systems the collapse load is higher than the yield load by the shape factor of the cross-section. This is because redistribution due to non-linear material behavior is possible only at cross-section

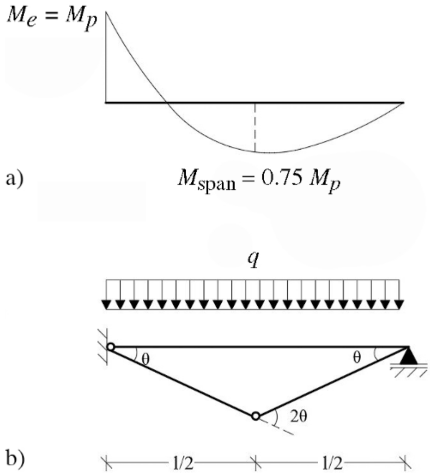


Fig. 2.14. Collapse load of a beam: a) assumed moment diagram, b) kinematic mechanism

level. On the contrary, statically in-determinate systems are more redundant, since distribution of internal forces and moments occurs at both cross-section and system level. Accordingly, more plastic hinges may form and the systems become more redundant. Redundancy is important in situations of structural overloading, as in cases of accidental actions such as explosions, fire, column loss, during earthquakes and generally when elements are stressed beyond their elastic limit. For more information on plastic analysis and relevant linear programming techniques reference is made in the literature [2.16].

2.9.3 Cross-section classification

Although moment-curvature curves are most easily determined analytically, they are extremely difficult to be confirmed by experimental methods since they require strain measurements at internal cross-section points. On the contrary, experimental evidence to inelastic cross-section behavior may be provided by three-point bending tests in which a concentrated force is applied at mid-span of a simply supported beam and the applied force and end-rotation recorded. As shown in Fig. 2.15 the shape of the resulting moment-rotation curves is similar for any kind of cross-sections due to the fact that the response is influenced by yielding, strain hardening and local buckling. However, the actual values of the moments and the rotations differ allowing thus a distinction to four cross-sections classes as following:

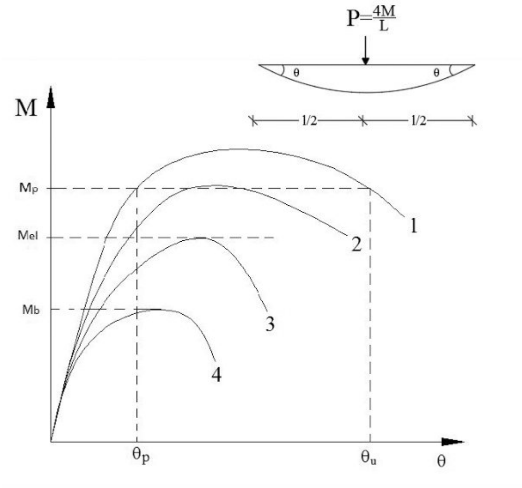
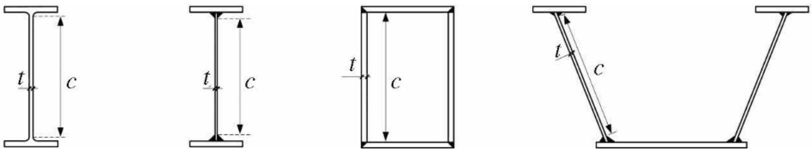


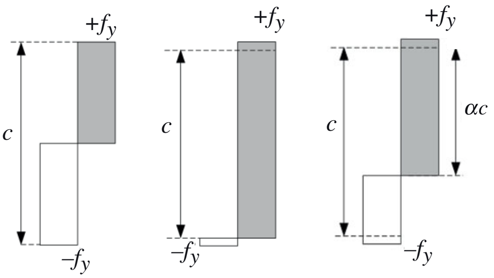
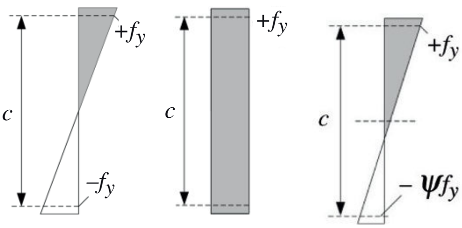
Fig. 2.15. Moment-rotation curves from three-point bending tests

- Class 1: Cross-sections develop the plastic moment and have sufficient rotation capacity for plastic hinges to form.
- Class 2: Cross-sections develop the plastic moment but have limited rotation capacity.
- Class 3: Cross-sections develop the elastic, yield, moment.
- Class 4: Cross-sections develop a limit moment smaller than the elastic moment due to early local buckling.

Local buckling of the cross-section plated walls plays for all cross-section classes an essential role. This is indicated by the fact that the curves for all classes have an unloading branch. However, the time of its appearance differs. For class 1 sections it appears at high strains where the material develops considerable strain hardening, while for class 4 sections at small elastic strains. Accordingly, cross-section walls are classified separately according to their loading and support conditions and the width over thickness (c/t) ratios. The loading conditions are expressed by the stress distribution over the width of the wall, while the support conditions by consideration of internal and external elements with two or one supporting edge. Finally, the cross-section has the least class, from 1 down to 4, of all individual walls. Based on experimental and numerical investigations, limit (c/t)-values were developed for the different classes [2.17] to [2.23]. Tables 2.7 and 2.8 provide the limit (c/t) values of internal and external elements. The background is summarized in [2.24]. Further

Table 2.7. Classification of internal elements

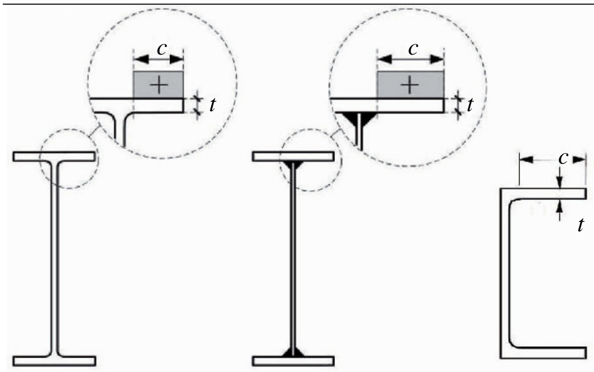


Class of part	*Part in bending*	*Part in compression*	*Part in bending and compression*
Stress distribution in relevant part (*compression positive*)			
1	$\frac{c}{t} \leqslant 72 \cdot \varepsilon$	$\frac{c}{t} \leqslant 33 \cdot \varepsilon$	$\alpha > 0.5 \Rightarrow \frac{c}{t} \leqslant \frac{396 \cdot \varepsilon}{13 \cdot \alpha - 1}$ $\alpha \leqslant 0.5 \Rightarrow \frac{c}{t} \leqslant \frac{36 \cdot \varepsilon}{\alpha}$
2	$\frac{c}{t} \leqslant 83 \cdot \varepsilon$	$\frac{c}{t} \leqslant 38 \cdot \varepsilon$	$\alpha > 0.5 \Rightarrow \frac{c}{t} \leqslant \frac{456 \cdot \varepsilon}{13 \cdot \alpha - 1}$ $\alpha \leqslant 0.5 \Rightarrow \frac{c}{t} \leqslant \frac{41.5 \cdot \varepsilon}{\alpha}$
Stress distribution in relevant part (*compression positive*)			
3	$\frac{c}{t} \leqslant 124 \cdot \varepsilon$	$\frac{c}{t} \leqslant 42 \cdot \varepsilon$	$\psi > -1 \Rightarrow$ $\frac{c}{t} \leqslant \frac{42 \cdot \varepsilon}{0.67 + 0.33 \cdot \psi}$ $\psi \leqslant -1 \Rightarrow$ $\frac{c}{t} \leqslant 62 \cdot \varepsilon \cdot (1 - \psi) \cdot \sqrt{(-\psi)}$

Notation

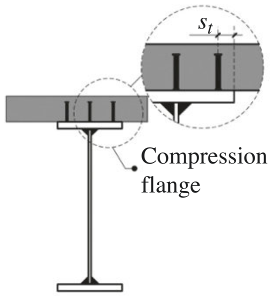
$\varepsilon = \sqrt{\frac{235}{f_y}}$ f_y in [MPa] for buckling analysis or $\varepsilon = \sqrt{\frac{235}{\sigma_{com}}}$ for section design
where σ_{com} is the maximum compression stress in the part in [MPa].

Table 2.8. Classification of external elements



Hot rolled Welded

Compression part attached to the concrete slab



Composite section

Class of part	*Part in compression*		*Compression part attached to the concrete slab*
1	Hot rolled	$\frac{c}{t} \leqslant 10 \cdot \varepsilon$	*Class* 1 If the centre-to-centre longitudinal spacing of the shear connectors is not greater than: $-22 \cdot \varepsilon \cdot t$ for slabs which are in contact with the top flange over the full length (e.g. solid slabs) $-15 \cdot \varepsilon \cdot t$ for slabs which are not in contact with the top flange over the full length (e.g. slab with profile steel sheeting; and $-s_t \leqslant 9 \cdot \varepsilon \cdot t$)
	Welded	$\frac{c}{t} \leqslant 9 \cdot \varepsilon$	
2	Hot rolled	$\frac{c}{t} \leqslant 11 \cdot \varepsilon$	
	Welded	$\frac{c}{t} \leqslant 10 \cdot \varepsilon$	
3	Hot rolled	$\frac{c}{t} \leqslant 15 \cdot \varepsilon$	
	Welded	$\frac{c}{t} \leqslant 14 \cdot \varepsilon$	

Notation
ε as Table in 2.7

proposals based on strain oriented effective width methods [2.25] are provided in [2.26] to [2.29]. It is emphasized that cross-section classification refers only to direct, normal, stresses σ. Resistance to shear stresses follows other rules that do not coincide with the cross-section classes.

Cross-section classification has implications in frame analysis and design. Plastic methods are allowed to be employed for analysis and design of frames or structural elements composed of class 1 cross-sections, at least at the positions where plastic hinges are expected to develop. For class 2 cross-sections, design may be plastic but analysis elastic, while for class 3 sections both analysis and design must be elastic. Finally, class 4 cross-sections must be verified to plate buckling. Obviously, elastic methods may be employed to all classes. Table 2.9 summarizes analysis and design methods for all classes.

Table 2.9. Methods of analysis and design for frame and beam element structures

Methods of analysis	*Methods of design*	*Failure criteria*	*Cross-section classes*
Elastic	Elastic + plate buckling	Plate buckling	4
Elastic	Elastic	Fiber failure	1, 2 or 3
Elastic	Plastic	Cross-section failure	1 or 2
Plastic	Plastic	System failure	1

2.9.4 Cross-section models for deformation controlled analyses

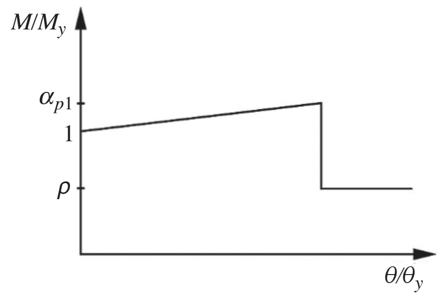


Fig. 2.16. Non-linear behavior of potential plastic hinges

Sometimes it is necessary to study the structural response to imposed deformations and not loading and accordingly to investigate the unloading branch of the equilibrium path. Typical examples are non-linear static analyses to seismic loading, called pushover analyses, or the examination of column loss in structural robustness studies, called pushdown analyses. In such analyses deformations of a specific pattern are imposed and gradually increased, those being horizontal for seismic loading or vertical for column loss. In such investigations, potential plastic hinges with specified non-linear behavior including both loading and unloading branches are defined. Fig. 2.16 shows a model for such a plastic hinge behavior. M_y is the yield moment, θ_y the yield rotation. For beams with double curvature as parts of moment resisting frames this can be set equal to $\theta_y = \frac{M_p \cdot l_b}{6EI_b}$, where M_p is the plastic moment, l_b the length and EI_b the bending stiffness of the beam. The parameters of the curve, such as the moment ratio at unloading ρ, or the rotation at which this takes place, depend on the type of member whether beam, column or brace and the class of cross-section.

2.10 Geometrically non-linear analysis (GNA)

2.10.1 Kinematic relations

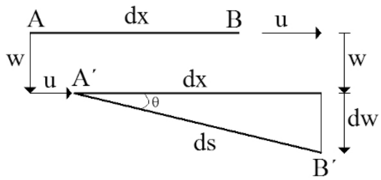


Fig. 2.17. Beam element before and after loading

The length of the beam element of Fig. 2.17 due to bending is equal to:

$$ds = \sqrt{dx^2 + dw^2} = \sqrt{1 + w'^2} \cdot dx \quad (2.19)$$

and its strain equal to:

$$\varepsilon = \frac{ds - dx}{dx} = \sqrt{1 + \theta^2} - 1 = \left(1 + \frac{1}{2} \cdot \theta^2 + \ldots\right) - 1 \approx \frac{1}{2} \cdot \theta^2 = \frac{1}{2} w'^2 \quad (2.20)$$

Accordingly, the total potential contains an additional term compared to 1st order theory as presented in Table 2.3 which is written as:

$$V = \int \left(\frac{1}{2} EI \cdot w''^2 \pm N \cdot \frac{1}{2} \cdot w'^2 - q \cdot w \right) dx \tag{2.21}$$

where $+$ applies when N is a tension force and $-$ when it is a compression force.

From equation (2.19) the curvature may be written as:

$$\kappa = -\frac{\dfrac{d\theta}{dx}}{\dfrac{ds}{dx}} = -\frac{\theta'}{(1+w'^2)^{1/2}} \tag{2.22}$$

Additionally, it is:

$$(\tan\theta)' = \left(\frac{\sin\theta}{\cos\theta}\right)' = \frac{\theta'}{\cos^2\theta} = \theta' \cdot (1+\tan^2\theta) \rightarrow \theta' = \frac{(\tan\theta)'}{1+\tan^2\theta} = \frac{w''}{1+w'^2} \tag{2.23}$$

Combining the above equations yields the kinematic condition for the curvature according to 3rd order theory:

$$\kappa = -\frac{w''}{(1+w'^2)^{3/2}} \tag{2.24}$$

The above kinematic relations may be extended when strains due to axial forces N are explicitly considered.

2.10.2 Analytical solutions

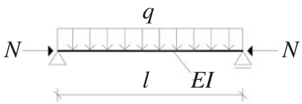
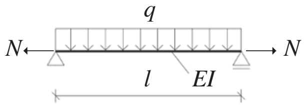
Analyses according to 2nd or 3rd order theories are performed in design practice by electronic methods using appropriate software. However, analytical solutions for specific simple cases have a certain value as they help understanding the behavior. Table 2.10 presents analytical solutions for a simply supported beam subjected to uniform transverse loading and a compression or tension axial force. It may be seen that an important parameter is the strut index ρ that expresses the loading conditions (N), the stiffness (EI) and the length (l) of the beam.

Fig. 2.18 presents the deflection at mid-span under increasing compression or tension axial load of such a beam, where the transverse loading is kept constant. It may be seen that both curves are non-linear, where deflections increase over- or under-proportionally when the beam is subjected to compression or correspondingly to tension. This shows that a tension force makes the beam response stiffer, as well known for cables, while a compression force leads to gradual stiffness loss and eventually to instability when a certain, critical, load is achieved.

2.10.3 Numerical solutions – Rayleigh/Ritz method

Numerical solutions in practical applications are based on FEM methods. However, other methods, like the energy based Rayleigh/Ritz method, have similarly to the

Table 2.10. Simply supported beam according to 2^{nd} order theory. Deformations and internal forces

	N compression	*N tension*
		
Deflection $w = \frac{q}{EI} \cdot \left(\frac{l}{\rho}\right)^4 \cdot$	$\left[\cos \rho\xi + \frac{1-\cos\rho}{\sin\rho} \cdot \sin \rho\xi - 1 - \frac{\rho^2}{2} \cdot (1-\xi) \cdot \xi\right]$	$\left[\frac{\sinh \rho\xi + \sinh\rho\xi'}{\sinh\rho} - 1 + \frac{\rho^2}{2} \cdot (1-\xi) \cdot \xi\right]$
Rotation $\theta = w'$ $\theta = \frac{q}{EI} \cdot \left(\frac{l}{\rho}\right)^3 \cdot$	$\left[-\sin \rho\xi + \frac{1-\cos\rho}{\sin\rho} \cdot \cos \rho\xi - \frac{\rho}{2} \cdot (1-2\xi)\right]$	$\left[\frac{\cosh \rho\xi - \cosh\rho\xi'}{\sinh\rho} + \rho \cdot \left(\frac{1}{2} - \xi\right)\right]$
Moment $M = q \cdot \left(\frac{l}{\rho}\right)^2 \cdot$	$\left[\cos \rho\xi + \frac{1-\cos\rho}{\sin\rho} \cdot \sin \rho\xi - 1\right]$	$\left[1 - \frac{\sinh \rho\xi + \sinh\rho\xi'}{\sinh\rho}\right]$
Shear force $V = q \cdot \left(\frac{l}{\rho}\right) \cdot$	$\left[-\sin \rho\xi + \frac{1-\cos\rho}{\sin\rho} \cdot \cos \rho\xi\right]$	$\left[-\frac{\cosh \rho\xi + \cosh\rho\xi'}{\sinh\rho}\right]$
Notation Strut index: $\rho = l \cdot \sqrt{N/EI}$ $\quad \xi = x/l$		

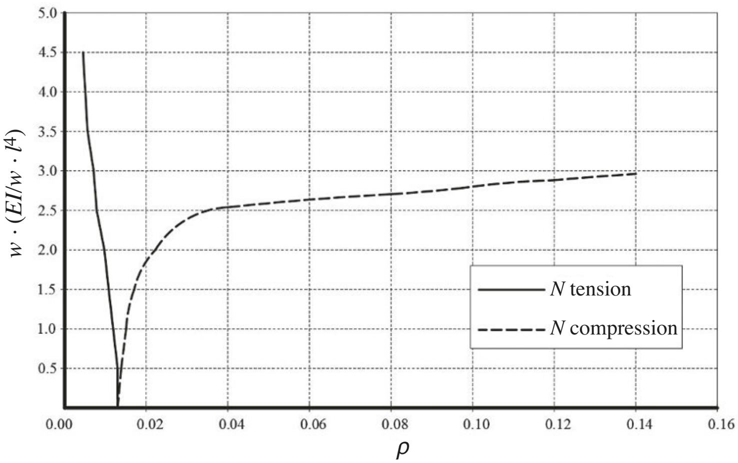


Fig. 2.18. Axial load-mid-span deflection of a beam under constant transverse loading according to 2^{nd} order analysis

analytic methods their value too. In the Rayleigh/Ritz method the deformations are defined by an approximate function $w = \Sigma a_i \cdot f_i$ that satisfies the geometric boundary conditions. This function is introduced in the equilibrium condition implying that the first variation of the total potential must be zero ($\delta V = 0$). The unknown parameters a_i are then determined from the conditions:

$$\frac{\partial(V)}{\partial a_i} = 0 \qquad (i = 1, 2 \ldots) \tag{2.25}$$

To illustrate the method, following approximate expression for the deflections of the beam of Fig. 2.19 satisfying the geometric boundary conditions $w(0) = w(1) = 0$ at its ends is selected:

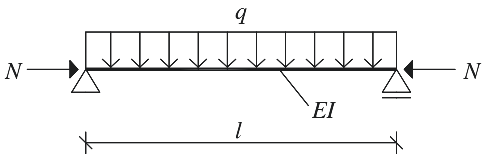


Fig. 2.19. 2^{nd} order analysis for a simply supported beam

$$w = a \cdot (\xi - \xi^2), \text{ with } \xi = x/l \tag{2.26}$$

Introducing this function in equation (2.21) of the total potential leads to following condition for its first variation:

$$\delta V = \frac{\partial}{\partial a} \int \left[\frac{1}{2} \cdot E \cdot I \cdot \frac{4 \cdot a^2}{l^4} - N \cdot \frac{1}{2} \cdot \frac{a^2 \cdot (1 - 2 \cdot \xi)^2}{l^2} - q \cdot a \cdot (\xi - \xi^2) \right] \cdot dx = 0 \tag{2.27}$$

which after derivation and integration leads to the unknown parameter α from:

$$a = \frac{q \cdot l^2}{6} \cdot \frac{1}{4 \cdot E \cdot I / l^2 - N/3} \tag{2.28}$$

Taking into account the strut index ρ, the final deflection is the determined from:

$$w^{II} = \cdot \frac{1}{1 - \rho^2/12} w^I \tag{2.29}$$

where:

w^I and w^{II} are the deflections according to 1^{st} and 2^{nd} order theory and
ρ is the strut index from Table 2.10.

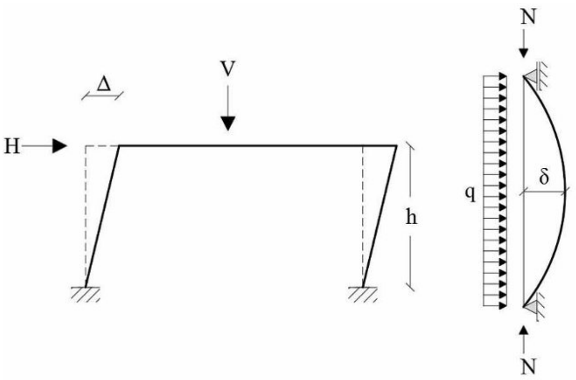
It may be seen that the 2^{nd} order deflection goes to infinity when $\rho = \sqrt{12} = 3.46$. The critical load is accordingly equal to $N_{cr} = \frac{3.46^2 \cdot E \cdot I}{l^2}$. Comparing this value to the true critical load $N_{cr} = \frac{\pi^2 \cdot E \cdot I}{l^2}$ confirms that the Rayleigh/Ritz method provides in general higher critical loads, unless the assumed deflection shape coincides with the true one. For further reading reference is made to the literature [2.30], [2.31].

2.10.4 Magnification factors for $P - \Delta$ and $P - \delta$ effects

The previous analysis indicates that deflections, or moments, according to 2^{nd} order theory may be determined by application of a magnification factor α to the corresponding values of the 1^{st} order theory.

Let's consider a typical story of a frame and a compression member as shown in Table 2.11 Bending moments M^I and horizontal displacements/deflections Δ^I/δ^I according to 1st order theory are only due to transverse loading. Considering equilibrium at the deformed state, the presence of vertical loads/ axial forces generates additional moments δM^1 and displacements/deflections $\delta\Delta^1/\delta\delta^1$. The additional moments/displacements/deflections are parts of a geometric sequence. Final moments and displacements/deflections are found by summation of the values determined at the individual steps. It is found that the final moments/ displacements/deflections from 2nd order theory may be determined by application of a magnification factor

Table 2.11. 2nd order theory for frame and member analysis. Magnification factors

	$P-\Delta$ *effects* *Sway buckling of frames*	$P-\delta$ *effects* *Non-sway buckling of members*
		
Iteration procedure for moments and deformations	$M^I = H \cdot h \to \Delta^I$ $\delta M^I = P \cdot \Delta^I \to \delta\Delta^1$ $\delta M^2 = P \cdot \delta\Delta^1 \to \delta\Delta^2$ ...	$M^I = f(q,l) \to \delta^I$ $\Delta M^1 = P \cdot \delta^I \to \delta\delta^1$ $\delta M^2 = P \cdot \delta\delta^1 \to \delta\delta^2$ ...
Additional moment of geometric sequence	$\delta M^n = M^I \cdot q^n$ $q = \frac{P}{P_{cr}} = \frac{1}{\alpha_{cr}}$ P_{cr} critical load of frame in sway mode	$\delta M^n = M^I \cdot q^n$ $q = \frac{P}{P_{cr}} = \frac{1}{\alpha_{cr}}$ P_{cr} critical load of member
Final 2nd order moments	$M^{II} = M^I + \delta M^1 + \delta M^2 + \ldots = \frac{M^I}{1-q} = \alpha \cdot M^I$	
Magnification factor for the moments	$\alpha = \frac{1}{1-\frac{P}{P_{cr}}}$ (2.30a)	
or general	$\alpha = \frac{1}{1-\frac{1}{\alpha_{cr}}}$ (2.30b) α_{cr} is the critical multiplier from LBA analysis	

α to the corresponding values from 1st order theory. This factor is a function of the critical multiplier α_{cr} from LBA analysis defined by equation (2.7).

Since horizontal floor displacements are defined as Δ, transverse member deflections as δ and axial loads as P, the corresponding effects in analysis are called $P-\Delta$ and $P-\delta$ effects, indicating the consideration of deformations in sway frame analysis or individual member analysis.

The critical multiplier for multi-story frames with sway buckling modes may by determined floor-wise from:

$$\frac{1}{a_{\mathrm{cr}}} = \theta = \frac{V_{Ed} \cdot \Delta}{H_{Ed} \cdot h} \tag{2.31}$$

where:

V_{Ed} is the total design vertical load at the top of the considered floor
H_{Ed} is the total design horizontal load at the base of the floor
Δ is the difference of horizontal 1st order displacements between top and bottom of the floor
h is the height of the considered floor
θ is the inter-story drift sensitivity coefficient, equal to the inverse of the critical multiplier.

It may be seen that the denominator of eq. (2.31), $H_{Ed} \cdot h$, is the overturning moment at the base of the floor of the un-deformed frame from 1st order theory, while the numerator, $V_{Ed} \cdot \Delta$, is the overturning moment due to sway 2nd order displacements.

The critical multiplier depends only on the vertical loading, V_{Ed}, and not on the horizontal loading, H_{Ed}, despite the fact that both loadings are introduced in eq. (2.31). Indeed, considering that the ratio H_{Ed}/Δ is the elastic stiffness, eq. (2.32) may be rewritten as:

$$\frac{1}{\alpha_{\mathrm{cr}}} = \frac{V_{Ed}}{k \cdot h} \tag{2.32}$$

where:

$k = H_{Ed}/\Delta$ is the elastic stiffness of the floor to sway displacements

Since H_{Ed} and Δ are proportional, k is independent on the level of horizontal loading but depends only on the geometric and inertial properties of the floor and its members.

Eq. (2.31) indicates that the critical state is reached when $\alpha_{\mathrm{cr}} = 1$. At that state the vertical loading has reached its critical value so that it is:

$$V_{\mathrm{cr}} \cdot \Delta = H_{Ed} \cdot h \rightarrow V_{\mathrm{cr}} = \frac{H_{Ed}}{\Delta} \cdot h \rightarrow V_{\mathrm{cr}} = k \cdot h \tag{2.33a}$$

Combining (2.32) and (2.33a) gives:

$$\frac{1}{\alpha_{\mathrm{cr}}} = \frac{V_{Ed}}{V_{\mathrm{cr}}} \tag{2.33b}$$

which confirms the validity of equation (2.32) for frames buckling in the sway mode.

1^{st} order analysis is generally allowed if its results do not deviate more than 10% from those of 2^{nd} order analysis, which means that it is allowed for values of the critical multiplier:

$$\alpha_{cr} > 10 \tag{2.34}$$

leading by introduction of (2.34) in (2.31) to magnification factors α smaller than 1.11, that complies with the above requirement since 2^{nd} order analysis moments are not higher than 10%, approximately, from 1^{st} order analysis moments.

For sway frames, and especially for seismic design, the corresponding requirement is written as a function of the inter-story sensitivity coefficient θ and not the critical multiplier. The provisions of the seismic code EN 1998-1-1 [2.32] are accordingly as following:

- 1^{st} order analysis is allowed when:

$$\theta \leqslant 0.1(\alpha_{cr} \geqslant 10) \tag{2.35a}$$

- Approximate 2^{nd} order analysis with use of magnification factors may be performed when:

$$0.1 < \theta \leqslant 0.2(10 > \alpha_{cr} \geqslant 5) \tag{2.35b}$$

- Exact 2^{nd} order analysis shall be performed when:

$$0.2 < \theta \leqslant 0.3(5 > \alpha_{cr} \geqslant 3.33) \tag{2.35c}$$

Values of $\theta > 0.3$ are not permitted for buildings in seismic areas. It should be mentioned that approximate 2^{nd} order analysis with use of magnification factors may be performed according to EN 1993-1-1 for single story frames when $\alpha_{cr} \geqslant 3$ (and not $\alpha_{cr} \geqslant 5$ which is stated in EN 1998-1-1).

2.11 Geometrically and materially non-linear analysis (GMNA)

This analysis takes into account both geometric and material non-linear behavior and requires for practical applications the availability of appropriate software. It provides directly ultimate limit loads for a structure or a structural element. For frame structures represented by beam elements, such an analysis is called 2^{nd} order plastic hinge or plastic zone analysis. However, different software programs may use other names. The user should look for which type of non-linear analysis the software is providing and how the various types are activated.

For hand calculations, 2^{nd} order rigid plastic analysis that neglects elastic deformations may be of interest since it provides the un-loading equilibrium path as shown in the example of Fig. 2.20 of a column under pure compression.

In rigid plastic analysis, elastic deformations are ignored so that the length of the column remains unchanged. The kinematic mechanism is built when a plastic hinge develops at mid-span, where the vertical distance between supports shortens because the column is not straight any more. The column shortens by the amount:

$$\Delta l = l \cdot (1 - \cos\theta) = l \cdot \left(1 - \sqrt{1 - \sin^2\theta}\right) = l \cdot \left(1 - \sqrt{1 - \theta^2}\right) \tag{2.36}$$

Ignoring higher order terms, it is $(1-\frac{\theta^2}{2})^2 = 1-\theta^2+\frac{\theta^4}{4} \approx 1-\theta^2$ so that finally:

$$\Delta l = l \cdot \theta^2/2 \tag{2.37}$$

Work by external load:

$$P_p \cdot \Delta l = P_p \cdot l \cdot \theta^2/2$$

Internal energy dissipation:

$$M_{pN} \cdot (2\theta)$$

where M_{pN} is the reduced plastic moment due to the presence of axial forces.

Equating the above relations and considering the geometric condition $w = \theta \cdot l/2$ gives the limit plastic load from:

$$P_p = 2 \cdot \frac{M_{pN}}{w} \tag{2.38}$$

Considering the interaction between axial forces and moments for I-sections that will be presented in chapter 3, the limit plastic load is determined from:

$$P_p = 2 \cdot \frac{M_p \cdot \left(1-\frac{P_p}{N_p}\right)/(1-0.5\cdot\alpha_w)}{w}$$

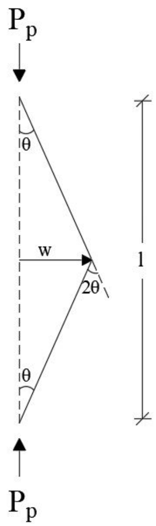


Fig. 2.20. Kinematic mechanism of a compression column

and after algebraic manipulation from:

$$P_p = \frac{2\cdot M_p}{2\cdot M_p/N_p + w\cdot(1-0.5\cdot\alpha_w)} \tag{2.39}$$

where:

M_p, N_p are the plastic moment and correspondingly axial force of the cross-section,
α_w is the ratio of the web area to the total cross-section area of the cross-section and
w is the deflection at mid-span.

Fig. 2.21 shows for an HEB 220 column from S 275 steel ($N_p = 2502$ kN, $M_p = 227$ kNm, $\alpha_w = 0.23$) the plastic limit load as a function of the deflection around the strong axis. It should be said that the curve of Fig. 2.21 has a theoretical character, since the very high values of the deflection that it predicts have a little physical sense.

2.12 Non-linear analyses with imperfections (GNIA, GMNIA)

This type of analysis is similar to the previous one, with the difference that imperfections, as presented in the next paragraph 2.13, are considered. For practical applications, appropriate analysis software should be available. However, hand calculations are possible for simple systems and are again of some value.

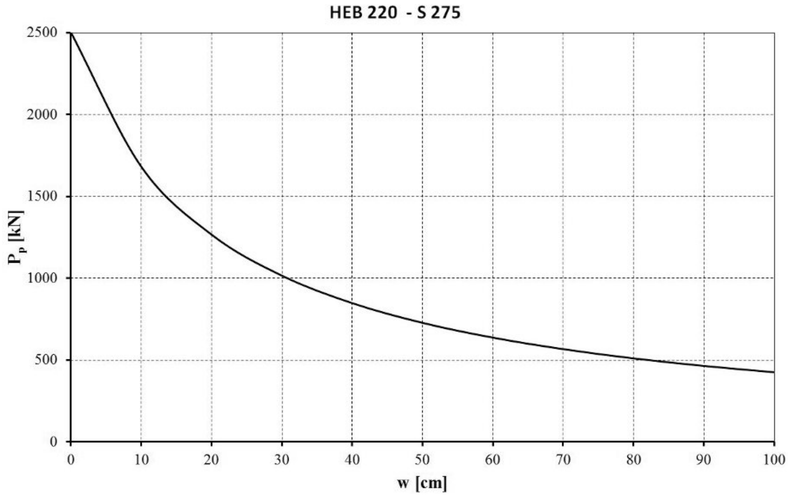


Fig. 2.21. Limit load of a column according to rigid plastic 2nd order analysis

Let's examine an axially loaded column of Fig. 2.22. The 2nd order deflection is according to Table 2.11 equal to $w = e_0 \cdot \frac{1}{1-P_e/P_{cr}}$. The ultimate elastic compression force is accordingly equal to:

$$P_e = P_{cr} \cdot (1 - e_0/w) \tag{2.40}$$

where:

e_0 is the bow imperfection
w is the deflection according to 2nd order theory and
P_{cr} is the elastic critical buckling, Euler, load.

Combining equations (2.39) and (2.40) the limit load may be determined. As an example, a simply supported compression column as examined in the previous paragraph, cross-section HEB 220, S 275 steel is considered. The length of the column is 10 m. A bow imperfection e_0 in the strong axis is considered equal to $e_0 = L/250 = 4$ cm, see Table 2.11. The load P_e is illustrated in Fig. 2.23 and intersects the curve from rigid plastic analysis at 1340 kN.

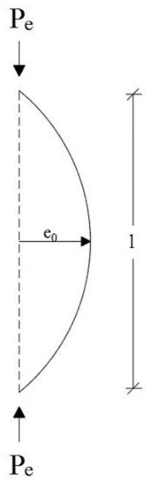


Fig. 2.22. Analysis of a compression column with a bow imperfection

A more conservative approximation for the limit load is provided by elastic-plastic 2nd order analysis according to the Rankine formula:

$$\frac{1}{P_u} = \frac{1}{P_e} + \frac{1}{P_p} \tag{2.41}$$

The results of such analysis are also illustrated in Fig. 2.23. The maximum load is 700 kN.

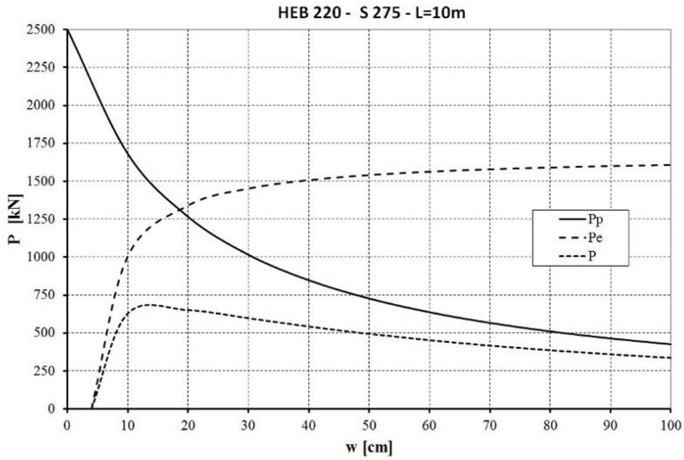


Fig. 2.23. Limit load of a column according to elastic and plastic 2nd order analysis

The major axis buckling load of this columns according to the provisions of Eurocode 3, chapter 5, is calculated as $P_b = 1170$ kN. Its value is between the two previously determined loads.

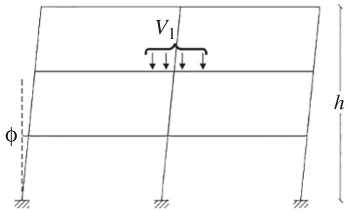
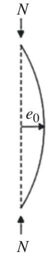
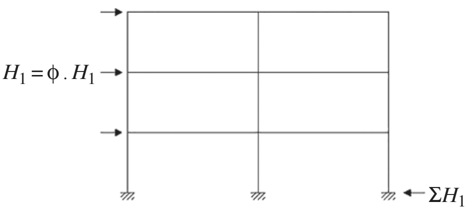
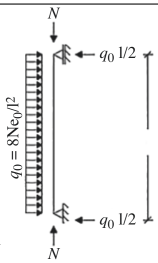
2.13 Imperfections in buildings

As explained in chapter 1, actual steel structures have for several reasons geometric and structural imperfections. Geometric imperfections are due to both fabrication and erection, while structural imperfections are due to fabrication, e.g. welding or cold forming. Accordingly, there are deviations between the nominal and the actual geometry of the structure as well as residual stresses in the material. The influence of imperfections is generally eroding the carrying capacity of structures and should be considered in analysis. Except in research or for special applications, both types of imperfections are usually merged for practical reasons to equivalent geometric imperfections.

Geometrical imperfections are distinguished in two types, sway imperfections and bow imperfections. Sway imperfections are considered in frame analysis, while bow imperfections are considered in individual columns, trusses or similar members. The two types of geometrical imperfections, as well as their actual values according to the EN 1993-1-1 [2.33] are illustrated in Table 2.12. Sway imperfections may be ignored when the design horizontal forces are high due to the fact that the building must be provided with sufficient stiffness and strength to resist the horizontal loads. Geometrical imperfections may be replaced by equivalent forces as shown in Table 2.12. To restore equilibrium, equivalent forces must be also set in the supports.

Sway imperfections must be always included in frame analysis, unless the horizontal design forces are higher than 15% of the vertical one according to equation (2.47), so that the effects of such imperfections are small. This is the case of frames subjected to high wind or seismic forces.

Table 2.12. Equivalent geometric imperfections to EN 1993-1-1 [2.33]

	Global sway imperfections	*Local bow imperfections*
		
Equivalent forces		
Values	$\phi = \phi_0 \cdot a_h \cdot a_m$ (2.42) $\varphi_0 = 1/200$ (2.43) $a_h = \frac{2}{\sqrt{h}}$ and $2/3 \leq a_h \leq 1$ (2.44) $a_m = \sqrt{0.5 \cdot \left(1 + \frac{1}{m}\right)}$ (2.45) $h =$ total height of building $m =$ number of columns in a row with compression force higher than 50% of the mean value of the compression for all columns in the frame.	$e_0 = l/j$ (2.46) where $j = 350, 300, 250, 200$ or 150 for elastic design of members associated with buckling curves a_0, a, b, c or d and $j = 300, 250, 200, 150$ or 100 for correspondingly plastic design of members
Imperfections may be omitted if condition (2.47) applies. Examples: Light single story buildings with wind as predominant loading, or multi-story buildings in regions of high seismicity.	$\frac{H}{V} \geq 0.15$ (2.47) H is sum of horizontal forces V is sum of vertical forces	• For tension members • For compression members in frames with a strut index: $\rho = l \cdot \sqrt{\frac{N}{EI}} < \frac{\pi}{2}$ (2.48) (or $\overline{\lambda} < 0.5 \cdot \sqrt{\frac{A \cdot f_y}{N}}$) $l =$ floor height

Local bow imperfections are generally neglected in frame analysis. However, members for which inequality (2.48) is **not** satisfied shall be provided with bow imperfections.

Care should be given to the fact that buildings are 3D-structures. Accordingly, global sway imperfections should be considered separately in each one direction and not simultaneously combined. This is valid also to member bow imperfections. They shall be considered separately in either of the two principal directions of the member cross-section.

For analysis of **bracing systems** such as wind braces in roofs of buildings, geometric bow imperfections are to be considered (Fig. 2.24):

$$e_0 = \alpha_m \cdot L/500 \tag{2.49}$$

with α_m from equation (2.45) and m equal to the number of members (frames) that are laterally braced.

This imperfection may be substituted by an equivalent force:

$$q = \sum_1^n N \cdot 8 \cdot \frac{e_0 + \delta}{L^2} \tag{2.50}$$

where:

δ is the horizontal brace deflection due to q and any direct horizontal forces from 1st order analysis. If 2nd order analysis is performed, δ is taken as zero (0).

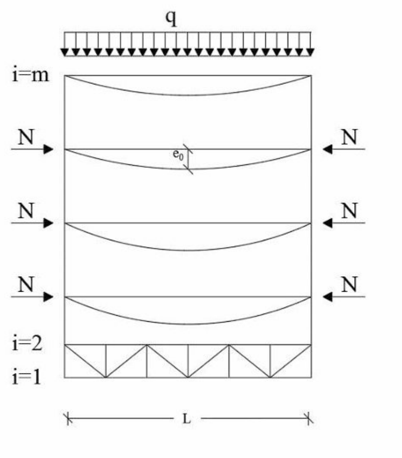


Fig. 2.24. Bracing systems imperfections and forces

For truss roofs the compression force N of Fig. 2.24 is the axial force of the top chord. For portal frames, it may be calculated from:

$$N = M/h \tag{2.51}$$

where:

M is the design moment at mid-span and
h is the girder depth.

Splices of columns or other compression members shall be designed for transverse forces at the splice position equal to:

$$H = 2 \cdot \varphi \cdot N \tag{2.52}$$

where:

φ is determined from equation (2.42) adopting a factor $\alpha_h = 1$.

For member verifications to **lateral torsional buckling** according to 2nd order theory, bow imperfections for the weak axis of the cross-section equal to $k \cdot e_0$ are to be considered. e_0 is the imperfection determined from equation (2.46), Table 2.11, where l is the length between lateral supports and k has a recommended value $k = 0.5$. For checking LTB of trusses, the direction of the bow imperfections is out-of-plane of the truss.

Imperfections may also be considered according to the general rule that they should follow the **shape of the elastic critical buckling mode** of the structure or element under consideration. Eurocode 3 provides that this buckling mode should be appropriately scaled by the factor f so that its amplitude be determined from:

$$\eta_{\text{init}} = f \cdot \eta_{\text{cr}} \tag{2.53}$$

where:

$$f = \frac{\alpha \cdot (\overline{\lambda} - 0.2)}{\overline{\lambda}^2} \cdot \frac{M_{Rk}}{EI\eta''_{\text{cr}\max}} \cdot \delta_e \tag{2.54}$$

$\delta_e = \frac{1-\frac{\chi\overline{\lambda}^2}{\gamma_{M1}}}{1-\chi\cdot\overline{\lambda}^2}$ factor transforming characteristic value to design value ($\delta_e = 1$ if $\gamma_{M1} = 1.0$)
α imperfection factor for the relevant buckling curve for critical member
χ reduction factor for the relevant buckling curve as a function of $\overline{\lambda}$ for critical member
$\alpha_{ult,k} = \min(\frac{N_{Rd,i}}{N_{Ek,i}})$, i refers to the members of the system
α_{cr} minimum load amplifier from LBA analysis to reach the fundamental critical buckling mode
$\overline{\lambda} = \sqrt{\frac{a_{ult,k}}{a_{\text{cr}}}}$
For framed structures with uniform members where i refers to all members

$$\overline{\lambda} = \max\left(\sqrt{\frac{A \cdot f_y}{a_{\text{cr}} \cdot N_{Ed}}}\right)_i$$

where i refers to all members and N_{Ed} is the axial force in the member
M_{Rk} characteristic moment resistance of the critical cross-section
N_{Rk} characteristic axial resistance of the critical cross-section

$EI\eta''_{\text{cr}\max}$ bending moment due to η_{cr} at the critical cross-section
η_{cr} shape of the elastic critical buckling mode

To the authors opinion this provision is too complicated and blurred for the practice. For building frames, they propose to scale the shape of the critical buckling modes in a way that the final global angle of sway ϕ in the 3D structure is equal to the one determined by equation (2.42), i.e. $\phi = \phi_0 \cdot a_h \cdot a_m$. Sway imperfections may be omitted if condition (2.47) holds.

An example on the application of this proposal is presented in Fig. 2.25. The 1$^{\text{st}}$ buckling mode indicates sway deformations of the front line of columns, while the 2$^{\text{nd}}$ sway deformations of the back line of columns. Since the two first critical load multipliers α_{cr} are too close, the two buckling modes should be combined to

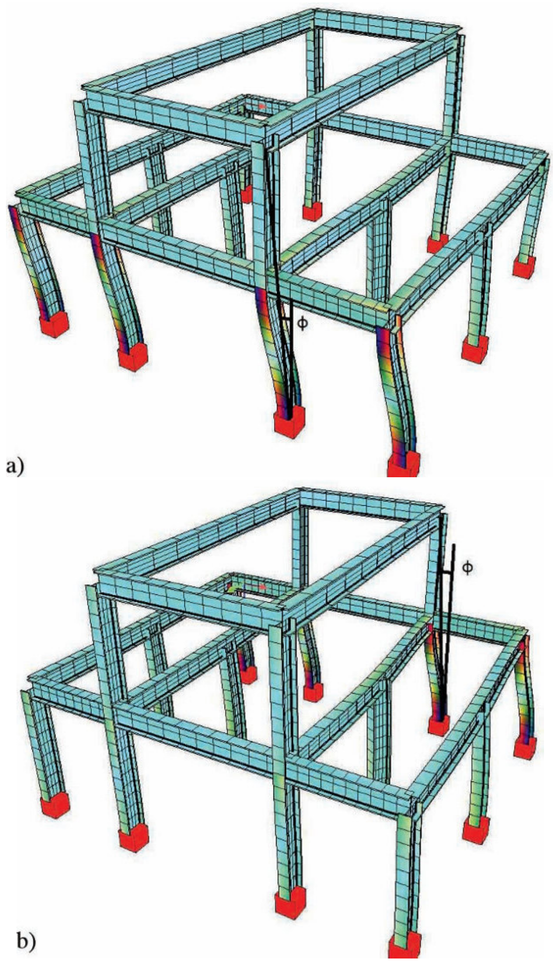


Fig. 2.25. Sway imperfections for a frame structure according to a) the 1$^{\text{st}}$ and b) the 2$^{\text{nd}}$ buckling modes

form the geometrical imperfections and scaled such that the imperfection angle ϕ is equal to what is calculated by equation (2.42). The advantages of this proposal are the following:

- Critical buckling modes detect soft story mechanisms and unlike EN 1993-1-1 lead to different angles for each story, despite the fact that the mean angle from the foundation to the top of the building is the same. The design is accordingly safer, as soft stories are assigned higher sway imperfections.
- Critical buckling modes detect weak directions of the buildings. For orthogonal lay-outs, these may be the principal directions, but for irregular ones they might be any.
- The proposal is simple and integrates smoothly with the existing rules of EN 1993-1-1.

It should be emphasized that although the above proposal for imperfections refers to building frames, it could be extended to other structural types where the critical buckling shapes are of bow shape. Imperfections in the shape of the critical buckling modes could be implemented and scaled such that the bow imperfection e_0 is equal to the member imperfection that corresponds to the relevant buckling curve, Table 4.2.

In case that local buckling modes of individual members are critical they shall be scaled such that the bow imperfection e_0 is equal to the member imperfection provided for the appropriate buckling curve, Table 4.2.

An example where the critical buckling mode corresponds to buckling of individual members is shown in Fig. 2.26. Here the bow imperfection for plastic design should be scaled to $e_0 = L/150$ if let's say the relevant buckling curve for the buckled member is European buckling curve *c*.

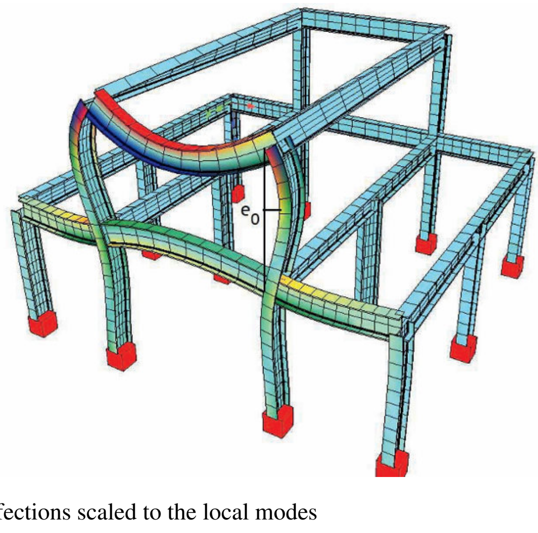


Fig. 2.26. Imperfections scaled to the local modes

2.14 Global analysis and design for building frames to Eurocode 3

Based on the provisions of EN 1993-1-1 several types of structural analysis and design may be performed for building frames that are represented by a complete 3D model. These types can be summarized as follows:

a) Method A
Geometrically non-linear 2nd order elastic global analysis including global and local imperfections, followed by elastic or plastic cross-section design. Three types of local imperfections have to be considered: bow imperfections of members for flexural buckling around the weak or strong principal axis and bow imperfections for lateral torsional buckling (LTB). The three types of imperfections do not need to be considered simultaneously. Checking directly LTB by analysis is possible if it is supported by the software, i.e. the beam elements employed must have at least 7 DOFs per node. Otherwise, if the usual 6DOF beam elements are employed, LTB member checks must be performed, or weak axis imperfections simulating LTB introduced.

b) Method B
Geometrically non-linear 2nd order elastic global analysis including sway imperfections, followed by cross-section and member design. Member design to flexural buckling is performed considering non-sway buckling lengths, i.e. the story height for column verifications. Buckling lengths for LTB design are determined in dependence of existing lateral supports. Local bow imperfections are covered by member design and need not be taken into account.
Method B may be subdivided in two types as following:
   - Method B_1
   Accurate 2nd order analysis is employed.
   - Method B_2
   Approximate 2nd order analysis is employed. This consists on performing 1st order analysis and magnification of sway effects by the amplification factor $\frac{1}{1-\frac{1}{1-\alpha_{cr}}}$.
   This method is applicable when $5 \leqslant \alpha_{cr} \leqslant 10$. For smaller values of α_{cr} method B_1 should be employed.
   For building structures in seismic areas values of $\alpha_{cr} < 3$ are not allowed.

c) Method C
Linear 1st order elastic analysis including sway imperfections, followed by cross-section and member design for the resulting internal forces and moments. Member design to flexural buckling is performed taking buckling lengths equal to the story height.

The alternative global analysis and design procedures for 3D building frames are summarized in Table 2.13.

Table 2.13. Procedures for global analysis and design for 3D building frames

		Imperfections		*Cross-section design*	*Member design Buckling length of columns L_{cr}*
Method	Analysis	Global sway imperfections	Local bow imperfections		
A	2^{nd} order full	Yes	Yes*	Yes	No
B_1	2^{nd} order accurate	Yes	No	Yes	Yes, L_{cr} = story height
B_2 Possible if $3^a \leqslant \alpha_{cr} \leqslant 10$ ^{a}5 in seismic areas	2^{nd} order approximate: 1^{st} order with amplification of sway effects by the factor $\frac{1}{1-1/\alpha_{cr}}$.	Yes	No	Yes	Yes, L_{cr} = story height
C Possible if $\alpha_{cr} \geqslant 10$	1^{st} order	Yes	No	Yes	Yes, L_{cr} = story height
Remarks	* Local bow imperfections shall be considered in respect to one or the other principal axes. For LTB checks, *k*-times weak axis local bow imperfections may be taken into account on the basis of the lateral bracing length, otherwise following the shape of the lowest LTB mode. Building frames with $\alpha_{cr} < 3$ are not allowed in seismic areas				

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3
Cross-section design

Abstract. During loading of the structure, cross-sections of structural members are subjected to internal forces and moments. This chapter provides the design resistances of cross-sections to individual internal forces and moments and their combinations. It starts with axial tension, where design resistances are given for cross-sections with or without holes and goes to the compression resistance of sections, accounting for possible local buckling effects. It then presents the elastic and plastic bending resistance, depending on the cross-section class, and the resistance to shear forces. Torsion and its uniform and non-uniform mechanisms with the corresponding design resistances are described. The properties and main characteristics in respect to torsion are given for open and hollow sections. Elastic and plastic resistances to St Venant and warping torsion are determined. Subsequently, cross-section design to combined internal forces and moments is given. Elastic design is expressed in terms of limitation of the von Mises stresses. For plastic design, interaction relations between internal forces and moments are defined including biaxial bending and axial force as well as shear forces and torsion. Interaction relations for plastic design of I-, H-, rectangular or circular hollow sections and angle sections as proposed by Eurocode 3 or derived by the authors are presented.

3.1 General

Structural analysis is followed by cross-section design, which verifies that the selected cross-sections of all members safely resist internal forces and moments for all loads and load combinations. Cross-section design at ultimate limit state (ULS) is indispensable for all methods of analysis employed. Later, it will be seen that cross-section design, in combination with geometric non-linear analysis checks also stability. Depending on the cross-section class, cross-sections are checked by elastic or plastic methods. In elastic design the most stressed cross-section fiber is checked against the limit stress, while in plastic design the capacity of the entire cross-section to form a plastic hinge is checked.

The limit stress is the ratio between a basic stress and a partial safety factor. The basic stress depends on the mode of failure and is:

- the yield strength f_y or
- the ultimate strength f_u

I. Vayas et al., *Design of Steel Structures to Eurocodes*, Springer Tracts in Civil Engineering, https://doi.org/10.1007/978-3-319-95474-5_3

The corresponding partial safety factors and their recommended values for buildings are:

- $\gamma_{M0} = 1.0$ and
- $\gamma_{M2} = 1.25$.

Accordingly, the limit direct stress is:

- f_y/γ_{M0} or
- f_u/γ_{M2}

The limit shear stress is generally equal to:

- $(f_y/\sqrt{3})/\gamma_{M0}$

3.2 Tension

A cross-section subjected to a tension force N applied to its centroid develops direct stresses σ and axial strains ε. Stresses and strains are uniformly distributed within the section for cross-sections without holes. The response to loading of a cross-section without holes is similar to the response of a coupon tensile test. For usual carbon steels three distinct regions may be distinguished (Fig. 3.1a):

- A linear elastic region with reversible deformations.
- A yield plateau region where the stress varies between the upper yield strength and the "static" yield strength. Most commonly, f_y is taken equal to the upper yield strength, otherwise conventionally as defined later.
- A strain hardening region where the stress increases up to the maximum attainable ultimate strength f_u in engineering terms, i.e. calculated on the basis of the initial cross-section.

The modulus of elasticity, E, is defined as the slope of the linear elastic region. The elastic region is associated with reversible deformations since unloading occurs with the slope E.

High strength steels exhibit throughout non-linear behavior without distinct yield plateau, so that their "yield" strength is defined conventionally as the stress that leads at unloading to 0.2% permanent strain.

The ductility ratio is defined as the ratio between the ultimate and the yield strain, as expressed by equation (3.1):

$$\mu = \frac{\varepsilon_u}{\varepsilon_y} \tag{3.1}$$

where:

ε_y is the yield strain and
ε_u is the ultimate strain that corresponds to f_u

Fig. 3.1a shows stress-strain curves for cross-sections with and without holes. Cross-sections without holes respond in a ductile manner similar to coupon tests. In cross-sections with holes stress concentrations around the hole appear from the beginning

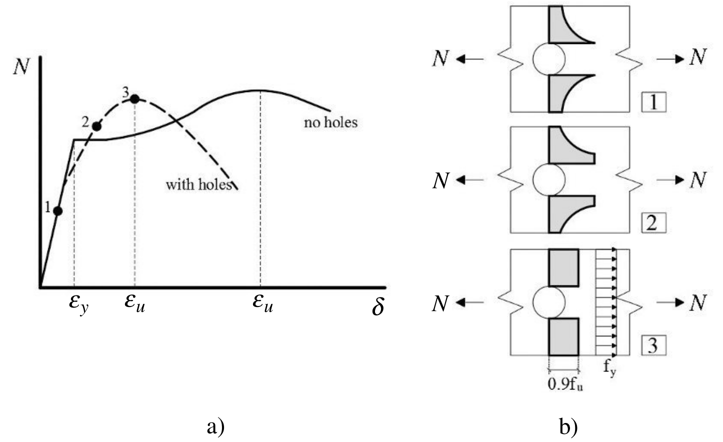


Fig. 3.1. a) Stress-strain curves for tension specimens with and without holes, b) stress distribution at various load levels

of loading (Fig. 3.1b). Such stress concentrations "yield out" at increasing loading as steel yields and strain hardens. At maximum load, stresses are almost uniformly distributed across the net section. The failure mode becomes brittle when the net section becomes much smaller than the gross section, i.e. when the number and diameter of holes in the section increase. This indicates that part of the material ductility is "consumed" in the redistribution of stresses in the net section. It may be seen that although steel is a ductile material, this does not necessarily apply to a steel structure. Steel structures must therefore be designed specifically for ductility, if this is required as for example for structures in seismic regions or when plastic analysis and design methods are employed.

A lot of experimental and numerical investigations have been performed to define design formulae for tension members without or with holes, staggered or not, some of which are listed in [3.1] to [3.7], which formed the basis for the design rules proposed by Eurocode 3 [3.8] as presented in the following.

The net area, A_{net}, is determined from the gross area, A, allowing for fastener holes, Fig. 3.2. The net area for a fracture line normal to the member axis, or the axis

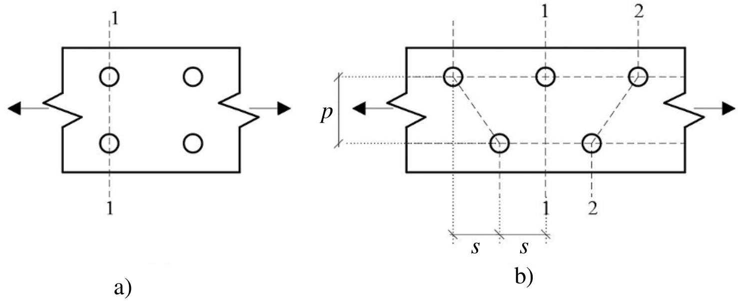


Fig. 3.2. Net section a) non-staggered, b) staggered fastener holes

of the axial force, is determined from following expression:

$$A_{net} = A - n \cdot d_0 \cdot t \quad (3.2)$$

where:

A is the gross cross-section area
n is the number of holes in the fracture line
d_0 is the diameter of the hole
t is the plate thickness

The net area for a zig-zag fracture line may be determined according to Cochrane's formula [3.1] from:

$$A_{net} = A - n \cdot d_0 \cdot t + m \cdot \frac{s^2 \cdot t}{4 \cdot p} \quad (3.3)$$

where:

p is the pitch between, centers of holes perpendicular to the axis of the member. For cross-sections with holes in more than one plane, p is measured along the middle axis of the cross-section, Fig. 3.3.
s is the spacing of the centers of two consecutive staggered holes parallel to the axis of the member.
m is the number of diagonals or zig zag lines connecting staggered holes and all other symbols as in eq. (3.2).

Fig. 3.2 shows examples of non-staggered and staggered bolt arrangements. For the non-staggered arrangement of Fig. 3.2a the net area is determined from:

$$\text{Section 1-1:} \qquad A_{net} = A - 2 \cdot d_0 \cdot t \quad (3.4)$$

For the staggered bolt arrangement of Fig. 3.2b the net area is the minimum of:

$$\text{Section 1-1:} \qquad A_{net} = A - d_0 \cdot t \quad (3.5)$$

$$\text{Section 2-2:} \qquad A_{net} = A - 2 \cdot d_0 \cdot t + \frac{s^2 \cdot t}{4 \cdot p} \quad (3.6)$$

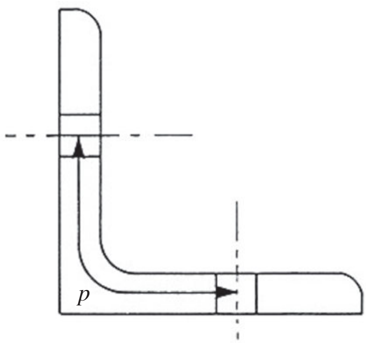


Fig. 3.3. Example of measuring distance p for angles with holes in both legs

The gross cross-section resistance to tension is associated with yielding and is determined from equation (3.7). Yielding defines “failure” in a conventional way, since it is not associated with fracture or collapse.

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \tag{3.7}$$

where:

A is the gross cross-section area
f_y is the yield strength of the material and
$\gamma_{M0} = 1.0$ is the partial safety factor for yielding.

The net cross-section resistance to tension is associated with fracture and is determined from:

$$N_{u,Rd} = \frac{0.9 \cdot A_{\text{net}} \cdot f_u}{\gamma_{M2}} \tag{3.8}$$

where:

A_{net} is the net cross-section area
f_u is the ultimate strength of the material and
$\gamma_{M2} = 1.25$ is the partial safety factor for fracture.

The tension resistance for angle sections connected through one leg must take into account the eccentricity between cross-section centroid where tension is applied and the centroid of the leg where it is resisted. Accordingly, the cross-section should be checked for tension and bending. This check may be avoided if the resistance to tension is determined from:

- for connection with one bolt (Fig. 3.4a):

$$N_{u,Rd} = \frac{2.0 \cdot (e_2 - 0.5d_0) \cdot t \cdot f_u}{\gamma_{M2}} \tag{3.9}$$

- for connection with two bolts (Fig. 3.4b):

$$N_{u,Rd} = \frac{\beta_2 \cdot A_{\text{net}} \cdot f_u}{\gamma_{M2}} \tag{3.10}$$

- for connection with three or more bolts (Fig. 3.4c):

$$N_{u,Rd} = \frac{\beta_3 \cdot A_{\text{net}} \cdot f_u}{\gamma_{M2}} \tag{3.11}$$

where

$\beta_2 = 0.4$ if $p_1 \leqslant 2.5d_0$
$\beta_2 = 0.7$ if $p_1 \geqslant 5.0d_0$
$\beta_3 = 0.5$ if $p_1 \leqslant 2.5d_0$
$\beta_3 = 0.7$ if $p_1 \geqslant 5.0d_0$ and
A_{net} is the net section of the angle profile.

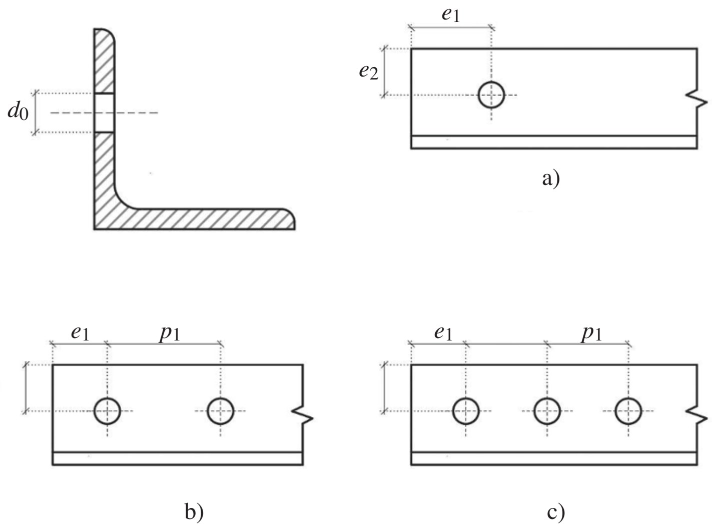


Fig. 3.4. Connection of angles through one leg with a) one, b) two, c) three or more bolts

For intermediate values of p_1, the coefficients β_2 and β_3 may be determined by linear interpolation.

For slip resistant connections of category C, see section 5.2.4, with high strength bolts the force is transmitted through friction so that the peak stress near the hole shown in Fig. 3.1b(1) does not develop. Accordingly, the cross-section resistance to tension corresponds to yielding in the net cross-section and is determined from:

$$N_{\text{ned},Nd} = \frac{A_{\text{net}} \cdot f_y}{\gamma_{M0}} \tag{3.12}$$

For ductile design of tension members, as for example in seismic resistant structures, yielding of the gross section must precede fracture at the net section. This ductility condition is written as:

$$N_{u,Rd} \geqslant N_{pl,Rd} \tag{3.13}$$

or considering equations (3.7) and (3.8):

$$\frac{A_{\text{net}}}{A} \geqslant \frac{(f_y/f_u)/(\gamma_{M2}/\gamma_{M0})}{0.9} \tag{3.14}$$

3.3 Compression

The resistance to compression for class 1, 2 or 3 cross-sections is due to yielding of the gross cross-section with no allowance for after fasteners to become fasteners holes, unless oversize or slotted holes are used. Accordingly, the cross-section compression capacity is determined from:

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \tag{3.15}$$

For class 4 cross-sections the influence of local buckling must be taken into account so that the compression resistance is determined from:

$$N_{c,Rd} = \frac{A_{\text{eff}} \cdot f_y}{\gamma_{M0}} \tag{3.16}$$

where:

A is the gross cross-section area and
A_{eff} is the effective cross-section area for compression, see section 2.9.3.

3.4 Bending

The bending capacity of cross-sections depends on the class of the cross-section. As outlined in 2.9.3, class 1 or 2 cross-sections develop the plastic moment as shown in Fig. 3.5.

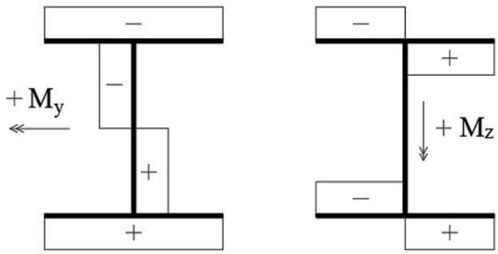


Fig. 3.5. Stress distribution at the plastic moment about *y-y* and *z-z* axis

The design bending resistance is accordingly equal to:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} \tag{3.17}$$

Class 3 cross-sections develop their elastic moment, i.e. the moment at which the most stressed fiber is reaching the yield stress, Fig. 3.6. The design bending resistance is given by equation (3.18). Class 1 or 2 cross-sections may of course be designed elastically, using the elastic moment instead of the plastic one.

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}} \tag{3.18}$$

Class 4 cross-sections develop a moment resulting from their effective section to bending. The design bending resistance is accordingly equal to:

$$M_{c,Rd} = \frac{W_{\text{eff}} \cdot f_y}{\gamma_{M0}} \tag{3.19}$$

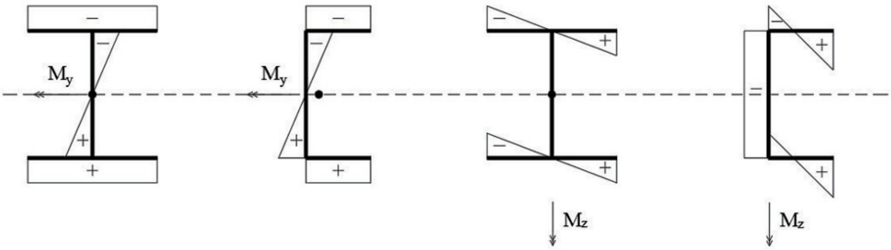


Fig. 3.6. Elastic stress distribution due to principal axes moments

In the above expressions (3.17) to (3.19):

W_{pl} ist the plastic section modulus,
W_{el} is the minimum elastic section modulus and
W_{eff} is the minimum effective section modulus.

The ratio $\alpha_{pl} = W_{pl}/W_{el}$ is called the shape factor since it depends only on the shape of the cross-section. Table 3.1 gives plastic moments and shape factors for various types of cross-sections.

In the determination of the cross-section properties, holes for fasteners in the tension zone may be ignored provided that condition (3.11) applies, where A_{net} and A refer to the tension zone that includes the tension flange and any part of the web under tension.

3.5 Shear force

Shear forces result in shear stresses which for elastic behavior may be determined at any point by the well-known formula from Engineering Mechanics:

$$\tau = \frac{V}{I}\frac{S}{t} \tag{3.20}$$

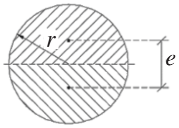
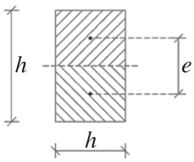
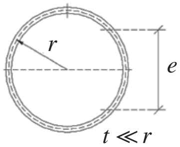
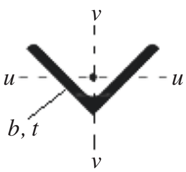
where:

V is the applied shear force
S is the first moment of area about the centroid axis of the portion of the cross-section between that point and the nearest boundary of the cross-section
I is the second moment of area of the cross-section around an axis perpendicular to the direction of the shear force and
t is the thickness at that point.

Fig. 3.7 shows shear stress distributions for several cross-section shapes. Following observations can be made:

a) Shear stresses develop not only for equilibrium conditions in direction of the applied shear force, but also perpendicular to it. The latter are not necessary to restore equilibrium with the applied shear force but develop due to the fact that shear resembles a “flow” which is the product between shear stress and wall thickness. At cross-section junctions, as e.g. between the flanges and the web,

Table 3.1. Plastic moments and shape factors for the usual types of cross-sections

	W_{pl}	W_{el}	α_{pl}
	$1.3333 \cdot r^3$	$\frac{\pi \cdot r^3}{4}$	1.70
	$0.25 \cdot b \cdot h^2$	$0.1667 \cdot b \cdot h^2$	1.50
	$4 \cdot t \cdot r^2$	$\pi \cdot t \cdot r^2$	1.27
			1.11-1.18
	u: $b^2 \cdot t/\sqrt{2}$	u: $b^2 \cdot t/\left(1.5 \cdot \sqrt{2}\right)$	1.5
	v: $b^2 \cdot t/\left(2 \cdot \sqrt{2}\right)$	v: $b^2 \cdot t/\left(3 \cdot \sqrt{2}\right)$	1.5

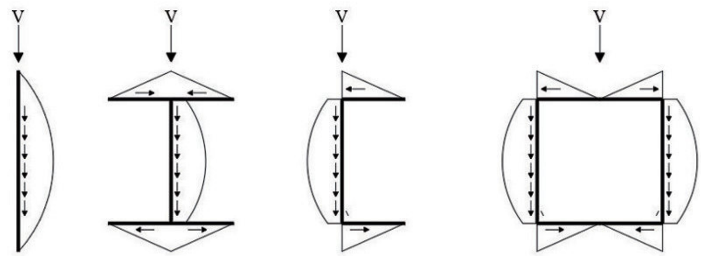


Fig. 3.7. Elastic shear stress distribution due to shear force

the incoming shear flow shall be equal to the outgoing flow, which results in:

- for T junctions as in Fig. 3.8:

$$\tau_w \cdot t_w = 2 \cdot \tau_f \cdot t_f \tag{3.21a}$$

and

- for L junctions:

$$\tau_w \cdot t_w = \tau_f \cdot t_f \tag{3.21b}$$

At edge points the stresses are zero because shear cannot "flow out" of the cross-section.

b) For cross-sections with flanges the stress distribution in the web becomes more uniform the stronger the flanges are. According to EN 1993-1-1 [3.8], in case the area of the flanges is much larger than the area of the web the entire shear force is resisted uniformly by the web. The relevant condition may be written as:

$$\tau = \frac{V}{A_w} \quad \text{if } A_f/A_w \geqslant 0.6 \tag{3.22}$$

where:

V is the applied shear force
A_f is the area of one flange
A_w is the area of the web that is the cross-section wall parallel to the force.

c) The applied shear forces refer to the shear center, not the centroid of the cross-section, so that they do not lead to torsion.

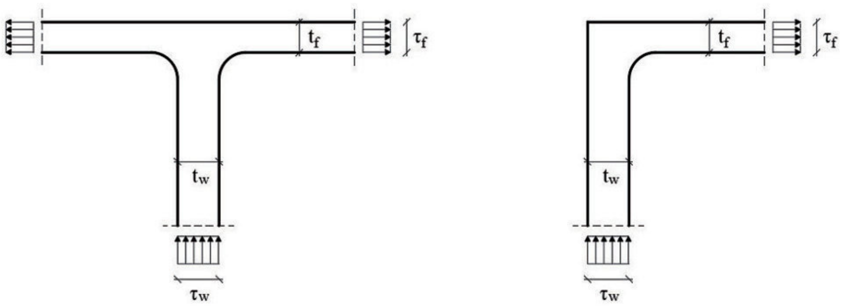


Fig. 3.8. Shear flow at T and L junctions

The shear center may be determined by the condition of zero torsion when shear forces apply through it. For example, the shear center of channel sections is not located in the web, but at a distance e from it which is determined by equation of the moments between the vertical and the horizontal pairs of forces shown in Fig. 3.9. The relevant condition writes (t = thickness of walls):

$$V \cdot e = H \cdot h \rightarrow e = \frac{H}{V} \cdot h = \frac{b^2 \cdot h^2 \cdot t}{4I_y} \tag{3.23}$$

For elastic design shear stresses determined from (3.20) or (3.22) are verified against the design shear strength according to the condition:

$$\tau_{Ed} \leqslant \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{3.24}$$

For plastic design, shear stresses are considered to be uniform along the shear area which includes the web and part of the flanges and be equal to the limit shear stress. The shear capacity is accordingly determined from:

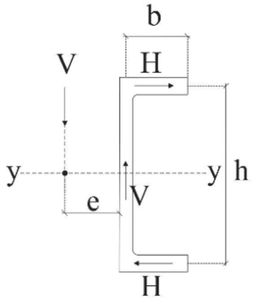


Fig. 3.9. Shear flow and shear center of a channel section

$$V_{pl,Rd} = A_v \cdot \frac{f_y/\sqrt{3}}{\gamma_{M0}} \tag{3.25}$$

where:

A_v is the shear area.

The plastic shear capacity as above may be reached when no shear buckling occurs. This is the case for unstiffened webs that fulfill the condition:

$$\frac{h_w}{t_w} \leqslant 72 \cdot \frac{\varepsilon}{\eta} \tag{3.26}$$

where:

$\varepsilon = \sqrt{\frac{235}{f_y}}$, f_y in MPa
$\eta = 1.20$ (recommended) for steel grades up to S 460
$\eta = 1.00$ (recommended) for steel grades higher than S 460.

For shear forces acting along the strong axis, Fig. 3.10, the shear area is determined from:

Rolled I and H sections:	$A_v = A - 2bt_f + (t_w + 2r)t_f \geqslant \eta \cdot h_w \cdot t_w$	(3.27a)
Rolled U sections:	$A_v = A - 2bt_f + (t_w + r)t_f$	(3.27b)
Rolled RHS of uniform thickness:	$A_v = Ah/(b+h)$	(3.27c)
Rectangular plates:	$A_v = A$	(3.27d)
Welded I, H, or box sections:	$A_v = \eta \cdot \sum(h_w \cdot t_w)$	(3.27e)
CHS and tubes:	$A_v = 2A/\pi$	(3.27f)

where:

A is the cross-section area and all other symbols as defined in Fig. 3.10.
RHS is an abbreviation for rectangular hollow section and CHS for circular hollow section.

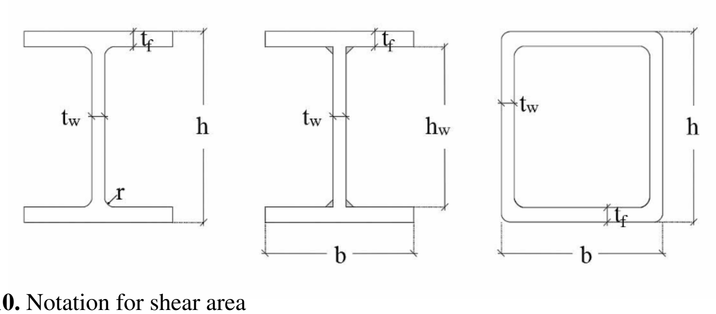


Fig. 3.10. Notation for shear area

For cross-sections with inclined webs, the vertical component of the inclined shear force must be taken into account so that it is:

$$A_v = h_{wv} \cdot t_w \cdot \cos\varphi \tag{3.28}$$

where ϕ is the angle of inclination of the web to the vertical.

For horizontal shear forces the flanges are parallel to the force and should be treated as "web", Fig. 3.11. The shear area must then be determined analogously.

Fastener holes need not be allowed for if the ductility condition (3.11) is fulfilled within the shear area, where γ_{M2} is substituted by γ_{M0} due to higher redistribution capacity for shear compared to tension.

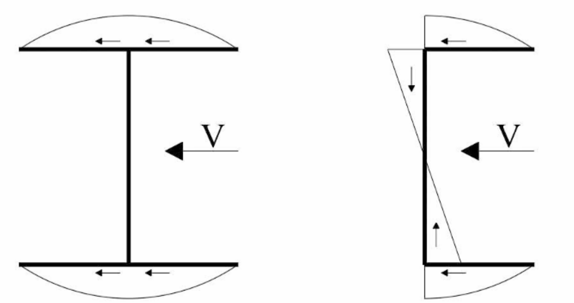


Fig. 3.11. Elastic shear stress distribution due to weak axis shear force

3.6 Torsion

3.6.1 General

Torsion is resisted by two mechanisms [3.9], [3.10]:

- Uniform or Saint Venant torsion and
- Non-uniform or warping torsion.

Accordingly, from equilibrium reasons it is:

$$M_t = M_{tp} + M_{ts} \tag{3.29}$$

where:

M_t is the total applied torsion moment
M_{tp} is the primary or St Venant torsion moment
M_{ts} is the secondary torsion moment due to warping.

The first mechanism of uniform or St Venant torsion results in the development of shear stresses, called primary shear stresses that "flow" along the cross-section walls. For closed sections this flow is uniform across the wall thickness, while for open sections shear stresses change sign within a cut of their walls since the net flow must be zero, Fig. 3.12. The torsional rigidity is then an order of magnitude higher for closed sections since the lever arm between the shear flows equals to the distance between opposite walls, while for open sections it is associated with the thickness of the walls that is very small for steel elements. The torsional, St Venant, rigidity is the product GI_t, where G is the shear modulus and I_t the torsional constant. It may be seen that the torsional rigidity is a function of G, not E, due to the fact that uniform torsion provokes shear and not direct stresses in the cross-section.

The torsion constant for open sections composed of flat walls with $b_i/t_i > 10$ is given by:

$$I_t = \frac{1}{3}\sum b_i \cdot t_i^3 \tag{3.30}$$

where:

i is the number of wall
b_i and t_i are the length and the thickness of the wall i.

The torsion constant for hollow sections is defined by the second Bredt formula and is equal to:

$$I_t = \frac{4A_0^2}{\oint \frac{ds}{t}} \tag{3.31}$$

where:

A_0 is the area enclosed by the middle line of the hollow section
ds is the elementary length of the wall and
t is the corresponding thickness of the wall.

For cross-sections with walls of constant thickness, the denominator may be expressed by a sum, so that it is:

$$\oint \frac{ds}{t} = \sum_{i=1,n} \frac{b_i}{t_i}$$

The torsion constants for the hollow sections of Fig. 3.12 are given by:

- Rectangular hollow sections (RHS):

$$I_t = \frac{4 \cdot (b_m \cdot h_m)^2}{\frac{2 \cdot (b_m + h_m)}{t}} = \frac{2 \cdot b_m \cdot h_m \cdot t}{\left(\frac{1}{h_m} + \frac{1}{b_m}\right)} \tag{3.32}$$

where:

$b_m = b - t$, $h_m = h - t$

- Circular hollow sections CHS:

$$I_t = \frac{4 \cdot (\pi \cdot r^2)^2}{\frac{2 \cdot \pi \cdot r}{t}} = 2 \cdot \pi \cdot r^3 \cdot t \tag{3.33}$$

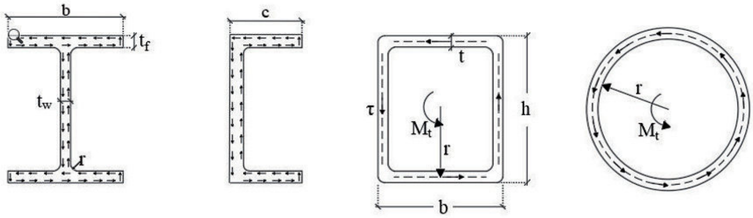


Fig. 3.12. Shear stress distribution due to torsion of open sections and hollow sections

In the second mechanism of non-uniform or warping torsion the applied torsion is resisted by a pair of in-plane transverse forces acting on the flanges, indicated as V_f in Fig. 3.13. Accordingly, the torsion resisted by this mechanism is equal to:

$$M_{ts} = V_f \cdot h \tag{3.34}$$

where:

h is the distance between flange mid-lines.

The flange forces V_f multiplied by the distance between the point of action of the torsion moment to the support lead to a pair of equal transverse flange bending moments M_f at the two flanges. These bending moments are of opposite sign, so that no net transverse bending moment develops for the entire cross-section. However, this pair of moments creates a further moment, of moments, called bimoment B due to the fact that it is a moment of moments and not forces. The bimoment is equal to:

$$B = M_f \cdot h \tag{3.35}$$

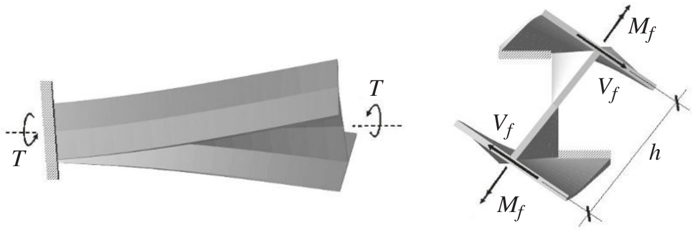


Fig. 3.13. Mechanism of warping torsion

The warping rigidity is expressed by the product EI_w, where E is the modulus of elasticity and I_w the warping constant. The modulus of elasticity is relevant for warping torsion due to the fact that the bimoment provokes direct stresses. For doubly symmetrical I-cross-sections the warping constant may be determined from eq. (3.36).

$$I_w = \frac{I_z h^2}{4} \tag{3.36}$$

where:

h is the depth of the cross-section, axial distance between flanges, and
I_z is the second moment of area about the weak axis.

For RHS sections the warping constant may be determined from eq. (3.37). For other cross-sections reference is made to the literature [3.10].

$$I_w = \frac{A_0^2 \cdot h \cdot t_w}{24} \cdot \left(\frac{\alpha - 1}{\alpha + 1}\right)^2 \cdot \left(\frac{b.t_f}{h \cdot t_w} + 1\right) \tag{3.37}$$

where, Fig. 3.12:

A_0 is the area defined in eq. (3.31)
h, b are the height and width of the cross-section
t_w, t_f the thicknesses of the corresponding walls and
$\alpha = \dfrac{b \cdot t_w}{h \cdot t_f}$

The split of torsion between the two mechanisms in the elastic region may be found by appropriate solution of the governing differential equation that is expressing the variation of the angle of twist along the member and depends on the ratio between the St Venant and the warping rigidities $\rho = \frac{G \cdot I_t}{E \cdot I_w}$. With increasing factor ρ, a larger part is resisted by uniform torsion, while for smaller values of ρ, warping torsion prevails. It is therefore clear that hollow sections resist torsion mainly by uniform torsion due to the high St Venant rigidity, while open sections by warping torsion. However, in general, the split of torsion between uniform and warping parts must be solved analytically considering the value of ρ. For numerical FEM calculations a solution is provided when 7-DOF or higher DOF beam elements are employed [3.11], [3.12]. Conventional 6-DOF beam elements are not able to include the effects of warping torsion and consider only St Venant effects. The problem can be also solved by application of the well-known analogy between warping torsion and 2nd order analysis of a beam under tension force and transverse loading for which reference is made to the literature [3.13].

Some open cross-sections have very small warping constant ($I_w \approx 0$) and cannot develop a warping torsion mechanism. Typical "warping free" cross-sections are illustrated in Fig. 3.13. It may be seen that in this category belong cross-sections that do not have two flanges, or where their walls meet at a common point, which is the shear center of the cross-section. These cross-sections are very susceptible to torsion and should not be used to resist applied torsion moments. Also some hollow sections have zero warping constant and do not develop warping. Eq. (3.37) indicates

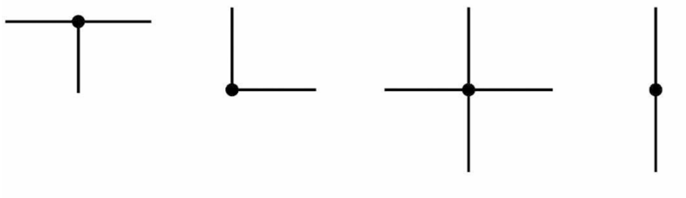


Fig. 3.14. Open cross-sections with $I_w = 0$

that this is the case when $\alpha = 1$. However, hollow cross-sections are beneficial in resisting torsion due to their high St Venant rigidity.

It should be mentioned that Eurocode 3 [3.8] allows hollow sections to resist torsion entirely by St Venant mechanism, while open section entirely by warping torsion.

3.6.2 Elastic design for torsion

3.6.2.1 St Venant torsion

For hollow sections of uniform thickness, Fig. 3.12, the shear stress due to uniform torsion is determined from:

$$\tau = \frac{M_t}{2 \cdot A_0 \cdot t} \tag{3.38}$$

where:

M_t is the applied torsion moment
A_0 is the area defined in eq. (3.31)
t is the wall thickness.

For open sections St Venant torsion may be neglected in practice. However, if it is considered, the maximal value of the resulting shear stress is determined from:

$$\max \tau = \frac{M_t}{I_t} \cdot t_{\max} \tag{3.39}$$

where:

M_t is the applied torsion moment
I_t is the torsion constant, eq. (3.30)
$t_{\max}$ is maximum thickness of all the walls.

Elastic design leads to the condition:

$$\max \tau \leqslant (f_y/\sqrt{3})/\gamma_{M0} \tag{3.40}$$

3.6.2.2 Warping torsion

Due to warping torsion direct and shear stresses, called secondary shear stresses, develop. The direct stresses are due to the bimoment B. They are illustrated in Fig. 3.15 and are determined from eq. (3.41):

$$\sigma_w = \frac{B}{I_w} \cdot \omega \qquad (3.41)$$

where:

ω is the warping function in m^2
B is the bimoment in kNm^2
I_w is the warping constant in m^6

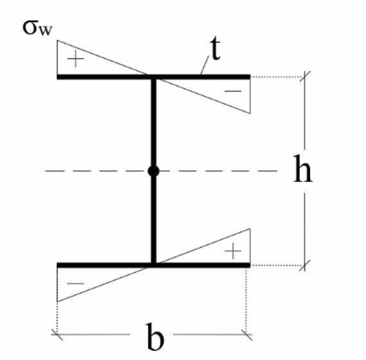


Fig. 3.15. Direct warping stresses σ_w

For I or H cross-sections with equal flanges eq. (3.41) may be written as:

$$\max \sigma_w = \pm \frac{B/h}{t \cdot b^2/6} \qquad (3.42)$$

where, Fig. 3.15:

h is the height of the cross-section (axial distance between flanges)
b is the flange width and
t is the flange thickness

Elastic design leads to the condition:

$$\max \sigma_w \leqslant f_y/\gamma_{M0} \qquad (3.43)$$

The transverse flange moments M_f lead to corresponding shear forces V_f and secondary shear stresses, Fig. 3.16, which may be determined from:

$$\tau_w = \frac{V_f}{I_{fz}} \frac{S_f}{t} \qquad (3.44)$$

V_f is the transverse flange shear force
S_f is the first moment of area defined in eq. (3.20)
I_{fz} is the second moment of area of flange about the weak axis of the beam and
t is the flange thickness.

The design for the shear forces is expressed by eq. (3.40).

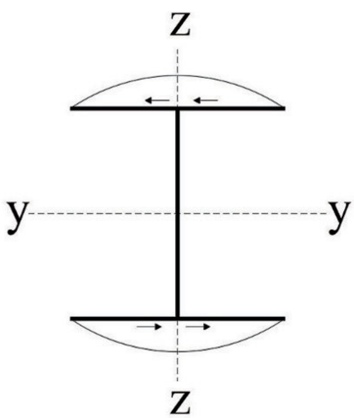


Fig. 3.16. Secondary shear warping stresses τ_w for open sections

3.6.3 Plastic design for torsion

3.6.3.1 St Venant torsion

For hollow sections, shear stress can be integrated over the thickness and the length of the walls to provide a resulting shear force $V_{T,Ed}$. This force is added to the corresponding force components caused in the walls due to the acting shear force. The total shear force in each wall resulting from shear and torsion shall be limited by the corresponding shear resistance of this wall $V_{pl,Rd}$ determined by equation (3.25).

For open sections the distribution of shear stresses due to St Venant torsion is such that the walls are in the full plastic condition as illustrated in Fig. 3.17.

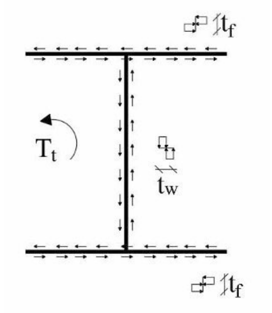


Fig. 3.17. Distribution of St Venant shear stresses at plastic state for open sections

The contribution of each wall to the torsional resistance is given by:

$$T_{t,pl,i,Rd} = (b_i \cdot t_i^2/4) \cdot \left(f_y/\sqrt{3}\right)/\gamma_{M0} \tag{3.45}$$

The total plastic St Venant torsion resistance of the cross-section is determined as the sum of the resistances of all the walls according to:

$$T_{t,pl,Rd} = \left(\frac{1}{4}\sum_i b_i \cdot t_i^2\right) \cdot \left(f_y/\sqrt{3}\right)/\gamma_{M0} \tag{3.46}$$

where:

b_i is the length (width) of the wall i
t_i is the thickness of the wall i

and the summation exerts over all the walls.

3.6.3.2 Warping torsion

As discussed before, warping torsion primarily develops in open sections. For such sections the plastic warping stress distribution due to bimoment B is illustrated in Fig. 3.18. The plastic bimoment $B_{pl,Rd}$ may be determined as following.

Doubly symmetrical I or H cross-sections

The plastic bimoment $B_{pl,Rd}$ may be determined from equation (3.47).

$$B_{pl,Rd} = M_{pl,z,Rd} \cdot h/2 \tag{3.47}$$

where:

$M_{pl,z,Rd}$ is the plastic moment of the cross-section about the weak axis z in kNm
h is the height of the cross-section (axial distance between flanges)

Simply symmetrical I or H cross-sections

The plastic bimoment $B_{pl,Rd}$ may be determined from equation (3.48).

$$B_{pl,Rd} = \min W_{pl,z,f} \cdot h \cdot f_y/\gamma_{M0} = \frac{b_u^2 \cdot t_u}{4} \cdot h \cdot \frac{f_y}{\gamma_{M0}} \tag{3.48}$$

where:

$\min W_{pl,z,f}$ is the minimum plastic section modulus between the two flanges
h is the height of the cross-section (axial distance between flanges)
b_u and t_u are the width and thickness of the weakest flange.

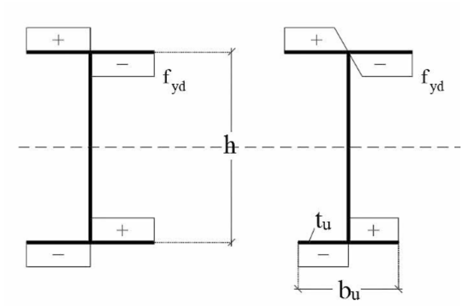


Fig. 3.18. Plastic stress distribution of warping stresses

3.7 Combination of internal forces and moments for elastic design

Elastic design may be performed at the level of stresses or at the level of internal forces and moments. For elastic FE analyses, where the structure or its parts are represented by plate, shell or volume elements design is performed checking the von Mises stresses at all structural points which should be limited to the design yield strength. For plane stress conditions, the relevant design criterion may be written as:

$$\sqrt{\sigma_{x,Ed}^2 + \sigma_{z,Ed}^2 - \sigma_{x,Ed} \cdot \sigma_{z,Ed} + 3 \cdot \tau_{Ed}^2} \leqslant \frac{f_y}{\gamma_{M0}} \tag{3.49}$$

where:

$\sigma_{x,Ed}$ and $\sigma_{z,Ed}$ are the design direct stresses along the axes x and correspondingly z
τ_{Ed} is the design shear stress.

In the above expression, the contribution of all internal forces and moments including St Venant and warping torsion is considered.

Elastic design may also be performed at the level of internal forces and moments, when analysis is made on the basis of a structural representation by beam elements. The design expression is represented by a linear interaction equation written as:

$$\frac{N_{Ed}}{N_{pl,Ed}} + \frac{M_{y,Ed}}{M_{el,y,Rd}} + \frac{M_{z,Ed}}{M_{el,z,Rd}} + \frac{B_{Ed}}{B_{el,Rd}} = 1 \tag{3.50}$$

where:

N_{Ed} is the design axial force
$M_{y,Ed}$, $M_{z,Ed}$ are the design moments along the strong and weak principal axes
B_{Ed} is the design bimoment
$N_{pl,Rd}$, $M_{el,y,Rd}$, $M_{el,z,Ed}$, $B_{el,Rd}$ are the corresponding design resistances.

The elastic bimoment resistance, $B_{el,Rd}$, is calculated by combination of equations (3.41) or (3.42) with (3.43).

Design expression (3.50) applies for class 3 cross-sections but may be used, although conservative, for class 1 and 2 cross-sections too. It is not valid for class 4 sections where the resistances are based on the properties of the effective cross-section [3.14], [3.15].

The influence of shear forces in the bending capacity may be ignored if it is:

$$\frac{V_{Ed}}{V_{pl,Rd}} \leqslant 0.5 \tag{3.51}$$

where:

V_{Ed} = design shear force from analysis
$V_{pl,Rd}$ = plastic shear resistance according to Eq. (3.25).

In the presence of torsion in hollow sections, V_{Ed} is calculated summing up algebraically the contribution of shear flows due to vertical or horizontal shear forces and St Venant torsion, see Fig.s 3.7, 3.11 and 3.12.

3.8 Combination of internal forces and moments for plastic design

3.8.1 Combination $N-M$ for rectangular cross-sections

The plastic resistances of a rectangular section shown in Fig. 3.19 are determined from:

$$N_{pl} = h \cdot t \cdot f_y \quad \text{and} \tag{3.52}$$

$$M_{pl} = h \cdot t^2 \cdot f_y/4 \tag{3.53}$$

The stress distribution indicates that for the acting forces and moments it is:

$$N = h_2 \cdot t \cdot f_y \quad \text{and} \tag{3.54}$$

$$M = h_1 \cdot (h - h_1) \cdot t \cdot f_y \tag{3.55}$$

By introduction of the geometric condition $h = 2 \cdot h_1 + h_2$ in the above relations, the final interaction relation is derived:

$$\left(\frac{N}{N_{pl}}\right)^2 + \frac{M}{M_{pl}} = 1 \tag{3.56}$$

The design expression is then written as:

$$\left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2 + \frac{M_{Ed}}{M_{pl,Rd}} \leqslant 1 \tag{3.57}$$

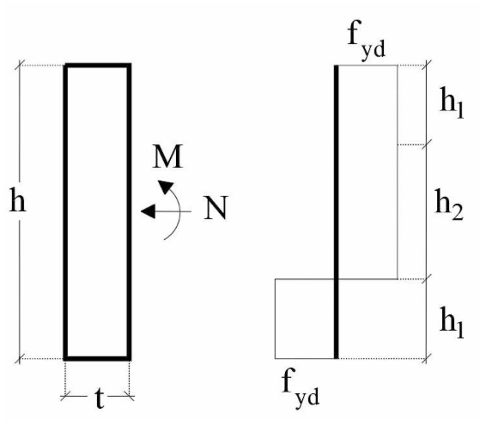


Fig. 3.19. Rectangular cross-section subjected to axial force and bending moment

The plastic moment resistance allowing for the presence of axial forces may be written as:

$$M_{N,pl,Rd} = M_{pl,Rd} \cdot \left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right] \tag{3.58}$$

The design expression writes then:

$$M_{Ed} \leqslant M_{N,pl,Rd} \tag{3.59}$$

3.8.2 Combination $N - M_y - M_z$ for a doubly symmetrical I cross-sections

3.8.2.1 Derivation of general formula

Interaction relations for combinations of internal forces and moments were derived by a number of authors, see [3.16] to [3.19], in which the question of whether to include the influence of warping bimoment in the interaction was a subject of heated discussions. In the following the procedure developed in [3.19], neglecting for simplicity the influence of warping bimoments, is presented and its results are compared with the relevant provisions of Eurocode 3 [3.23]. For further reading reference is made to the literature [3.20] to [3.22].

The cross-section is represented by the centroid axes of its walls as in Fig. 3.20. This idealization is exact for welded profiles provided that the length of the walls is sufficiently larger than the wall thickness. For rolled profiles it neglects the additional area provided by the existence of rounded corners at the flange-web junctions.

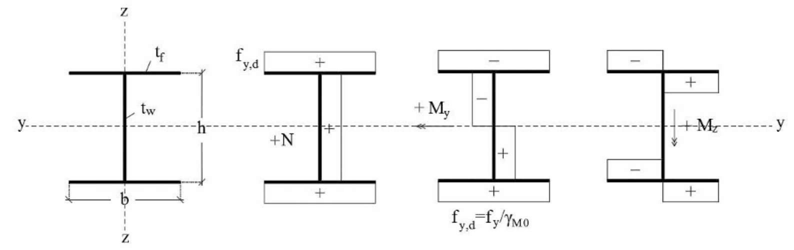


Fig. 3.20. Geometric idealization and stress distribution at plastic states for I cross-sections

The stress distribution in the cross-section at the full plastic state is illustrated in Fig. 3.20. The design plastic resistances are given by:

$$N_{pl,Rd} = A \cdot f_{y,d} \tag{3.60}$$

$$M_{pl,y,Rd} = W_{pl,y} \cdot f_{y,d} \tag{3.61}$$

$$M_{pl,z,Rd} = W_{pl,z} \cdot f_{y,d} = 1.5 \cdot W_{el,z} \cdot f_{y,d} \tag{3.62}$$

where:

$f_{y,d} = f_y/\gamma_{M0}$ is the design yield stress.

The corresponding full plastic resistances N_{pl}, $M_{pl,y}$, $M_{pl,z}$ are derived from the above expression setting f_y instead of $f_{y,d}$ $(= f_y/\gamma_{M0})$.

The geometrical properties of the cross-section may be written as following:

$$A = A_f + A_w \qquad \text{cross-section area:} \tag{3.63}$$

where:

$A_f = 2 \cdot b \cdot t_f$ is the area of the flanges and
$A_w = h \cdot t_w$ is the area of the web

The ratios of the flange, correspondingly web, area to the total area are:

$$\alpha_f = A_f/A \tag{3.64a}$$

$$\alpha_w = A_w/A = 1 - \alpha_f \tag{3.64b}$$

The non-dimensional partial axial force capacities of the flanges and correspondingly the web are:

$$n_f = \frac{N_{pl,f}}{N_{pl}} = \frac{N_{pl,f,Rd}}{N_{pl,Rd}} = \alpha_f \tag{3.65a}$$

$$n_w = \frac{N_{pl,w}}{N_{pl}} = \frac{N_{pl,w,Rd}}{N_{pl,Rd}} = \alpha_w = 1 - \alpha_f \tag{3.65b}$$

The bending moment capacities around the major axis y-y for the flanges and the web are:

$$M_{pl,y,f} = \frac{\alpha_f}{2} h \cdot A \cdot f_y \tag{3.66a}$$

$$M_{pl,y,w} = \frac{h^2 \cdot t_w}{4} f_y = \frac{A_w \cdot h}{4} f_y = \frac{\alpha_w}{4} h \cdot A \cdot f_y \tag{3.66b}$$

The sum of the two gives the plastic moment of the complete cross-section:

$$M_{pl,y} = M_{pl,y,f} + M_{pl,y,w} = \frac{1+\alpha_f}{4} h \cdot A \cdot f_y \tag{3.66c}$$

The flange and web contributions to the bending resistance in non-dimensional form may be determined by division of the individual capacities to the plastic moment:

$$m_{y,f} = \frac{M_{pl,y,f}}{M_{pl,y}} = \frac{M_{pl,y,f,Rd}}{M_{pl,y,Rd}} = \frac{2 \cdot \alpha_f}{1+\alpha_f} \tag{3.67a}$$

$$m_{y,w} = \frac{M_{pl,y,w}}{M_{pl,y}} = \frac{M_{pl,y,w,Rd}}{M_{pl,y,Rd}} = \frac{\alpha_w}{1+\alpha_f} = \frac{1-\alpha_f}{1+\alpha_f} \tag{3.67b}$$

The design internal forces and moments N_{Ed}, $M_{y,Ed}$, $M_{z,Ed}$ may be represented in non-dimensional form by division with the corresponding design plastic resistances, where the design forces and moments with the index Ed are considered with absolute, positive, values:

$$n = N_{Ed}/N_{pl,Rd} \tag{3.68}$$

$$m_y = M_{y,Ed}/M_{pl,y,Rd} \tag{3.69}$$

$$m_z = M_{z,Ed}/M_{pl,z,Rd} \tag{3.70}$$

Under the action of an axial force N_{Ed}, or n in non-dimensional form see eq. (3.68), the plastic state may be considered to start from the web until n reaches n_w see eq. (3.65b). For further loading, the plastic state starts at the junction between flange and web and extends to the edges of the flanges. The portion of the flange that is under the plastic state, i.e. the length of the flange into the plastic state divided by the total flange width as shown in Fig. 3.21a is designated as λ_N and is equal to:

$$\lambda_N = \frac{n - n_w}{\alpha_f} = \frac{n - \alpha_w}{\alpha_f} \quad \text{for} \quad n > n_w = \alpha_w \tag{3.71}$$

The cross-section resistance is fully exploited when the entire flange is into the plastic state. This is equivalent to $\lambda_N = 1$, or using eq. (3.71) $n = \alpha_w + \alpha_f = 1$.
Under the action of a bending moment $M_{y,Ed}$, or m_y in non-dimensional form see eq. (3.69), the plastic state starts from the web until m_y reaches $m_{y,w}$ see eq. (3.67b) and extends at further loading from the flange-web-junction to the edges of the flanges. The portion of the flange that is under the plastic state, Fig. 3.21b, is designated as λ_y and is equal to:

$$\lambda_y = \frac{M_y - M_{pl,y,w}}{M_{pl,y,f}} = \frac{M_y - M_{pl,y,w}}{M_{pl,y,}} \cdot \frac{M_{pl,y}}{M_{pl,y,f}} = \frac{m_y \cdot (1+\alpha_f) - \alpha_w}{2 \cdot \alpha_f} \quad \text{for} \quad m_y > m_{y,w} \tag{3.72}$$

The cross-section resistance is fully exploited when the entire flange is into the plastic state. This is equivalent to $\lambda_y = 1$, or using eq. (3.72) when

$$m_y = \frac{2 \cdot \alpha_f + \alpha_w}{1 + \alpha_f} = \frac{\alpha_f + 1}{1 + \alpha_f} = 1$$

Under the action of a bending moment $M_{z,Ed}$, or m_z in non-dimensional form see eq. (3.70), the plastic state starts from the edges of the flanges and extends to the flange-web-junction. The portion of the flange that is under the plastic state, see Fig. 3.21c, is designated as λ_z and is divided equally between the two edges. This portion is equal to:

$$\lambda_z = m_z \tag{3.73}$$

At the full plastic state it is $\lambda_z = m_z = 1$.

Under the simultaneous action of an axial force N_{Ed}, or n, and a bending moment $M_{y,Ed}$, or m_y, the sequence of plasticizing depends on whether the axial force is smaller than the axial force capacity of the web or not. If the former is the case, part of the bending moment may be resisted by the web. This part may be determined by application of eq. (3.58) because the web is a rectangular cross-section. Accordingly, when $n \leqslant n_w = \alpha_w$ the moment resisted by the web is equal to:

$$m_{N,y,w} = m_{yw} \cdot \left[1 - \left(\frac{n}{n_w}\right)^2\right] = \frac{1-\alpha_f}{1+\alpha_f}\left[1 - \left(\frac{n}{1-\alpha_f}\right)^2\right] = \frac{(1-\alpha_f)^2 - n^2}{1-\alpha_f^2} \tag{3.74}$$

In the second case, when $n > n_w = \alpha_w$, the web is fully exploited from the axial force so that the bending moment is resisted from the flanges only. In this case it is:

$$\lambda_y = \frac{My}{M_{pl,y,f}} = \frac{M_y}{M_{pl,y}} \cdot \frac{M_{pl,y}}{M_{pl,y,f}} = \frac{m_y}{m_{yf}} = \frac{m_y \cdot (1+\alpha_f)}{2 \cdot \alpha_f} \tag{3.75}$$

The portion of the flanges that resist the bending moment λ_{Ny} is equal to λ_y, eq. (3.72), when $n \leqslant n_w = \alpha_w$ and $\lambda_N + \lambda_y$, eq. (3.75) and (3.71), in the opposite case. After algebraic manipulation the resulting expressions may be written as:

$$\text{For} \quad n \leqslant n_w = \alpha_w \qquad \lambda_{Ny} = \frac{m_y \cdot (1+\alpha_f) - \alpha_w - n^2/\alpha_w}{2 \cdot \alpha_f} \tag{3.76}$$

$$\text{For} \quad n > n_w = \alpha_w \qquad \lambda_{Ny} = \frac{m_y \cdot (1+\alpha_f) - 2 \cdot (\alpha_w - n)}{2 \cdot \alpha_f} \tag{3.77}$$

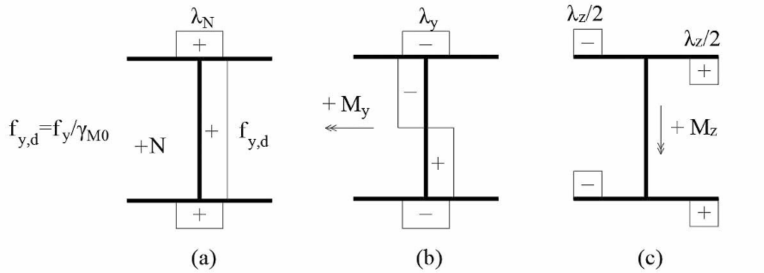


Fig. 3.21. Portions of the flanges that are in the plastic state

Under the simultaneous action of N_{Ed} and both bending moments $M_{y,Ed}$, $M_{z,Ed}$ the flanges are subjected in the portion λ_{Ny} from an axial force and in the portion λ_z from a bending moment for which it is:

$$\lambda_z = 1 - (\lambda_N^2 + \lambda_y^2) \tag{3.78}$$

The interaction expression is finally written as:

- For $n \leqslant \alpha_w$: $\left\{ \dfrac{m_y \cdot (1+\alpha_f) - \alpha_w + n^2/\alpha_w}{2 \cdot \alpha_f} \right\}^2 + m_z \leqslant 1$ (3.79a)
- For $n > \alpha_w$: $\dfrac{\left(m_y \cdot (1+\alpha_f)\right)^2 + \left(2(\alpha_w - n)\right)^2}{(2 \cdot \alpha_f)^2} + m_z \leqslant 1$ (3.79b)

with the additional condition: $m_y \leqslant \dfrac{1-n}{1-0.5 \cdot \alpha_w}$ (3.79c)

The above relations (3.79) may be simplified for special cases as presented in the following.

3.8.2.2 Combination N, M$_y$

In this case it is $m_z = 0$ and eq. (3.79) writes:

$$\text{For} \quad n \leqslant \alpha_w \qquad m_y \leqslant m_{Ny} = \frac{1 - \alpha_f^2 - n^2}{1 - \alpha_f^2} \tag{3.80a}$$

$$\text{For} \quad n > \alpha_w \qquad m_y \leqslant m_{Ny} = \frac{1-n}{1-0.5 \cdot \alpha_w} \tag{3.80b}$$

For the special case of a rectangular section it is $\alpha_f = 0$ and $\alpha_w = 1$, so that only eq. (3.80a) applies. This is written as $m_{Ny} = 1 - n^2$ which is identical to eq. (3.58).

For the special case of a cross-section composed only of two flanges it is $\alpha_f = 1$ and $\alpha_w = 0$, so that only eq. (3.80b) applies. This is then written as $m_{Ny} = 1 - n$ which is a linear interaction relationship. The two curves are illustrated in Fig. 3.22.

The interaction relations proposed by Eurocode 3 [3.23] for I-sections as expressed by eq. (3.81) are almost identical to (3.80):

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5a) \quad \text{but} \quad M_{N,y,Rd} \leqslant M_{pl,y,Rd} \tag{3.81}$$

where:

$n = N_{Ed}/N_{pl,Rd}$
$a = (A - 2bt_f)/A$, but $a \leqslant 0.5$

and all acting forces and moments are represented by positive values:

- In eq. (3.81) $a = \alpha_w$ of eq. (3.80) for the idealized section of Fig. 3.20.

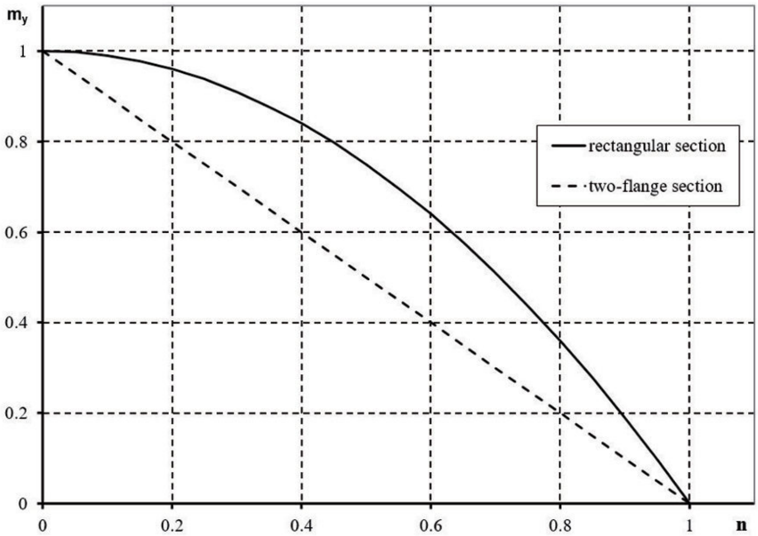


Fig. 3.22. Interaction diagrams N-M_y for a rectangular and a two-flange cross-section

Fig. 3.23 illustrates interaction diagrams representing eq. (3.81) for two profiles. It may be seen that the bending capacity is not reduced for small axial forces that may be resisted by the web alone.

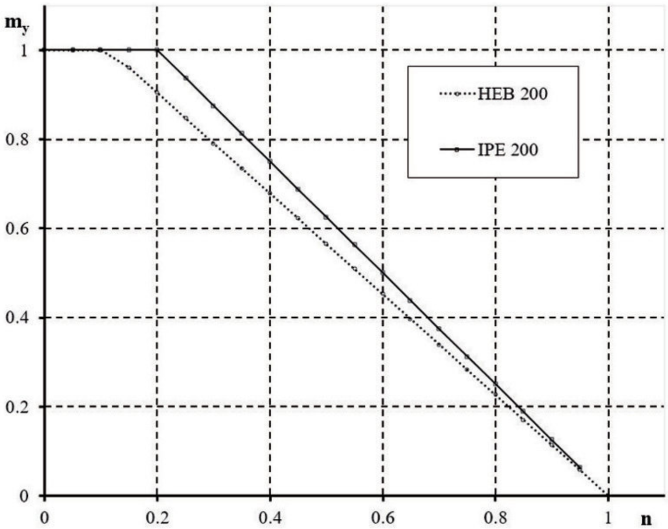


Fig. 3.23. Interaction diagrams N-M_y to EN 1993-1-1 [3.23] for strong axis bending with axial force

3.8.2.3 Combination N, M_z

Weak axis bending is resisted by the flanges alone due to the fact that the web is on the neutral axis and is not stressed by weak axis moments. In this case it is $m_y = 0$ and eq. (3.79) writes:

$$\text{For} \quad n \leqslant \alpha_w \qquad m_z \leqslant m_{Nz} = 1 \tag{3.82a}$$

$$\text{For} \quad n > \alpha_w \qquad m_z \leqslant m_{Nz} = 1 - \left(\frac{n - \alpha_w}{1 - \alpha_w}\right)^2 \tag{3.82b}$$

The corresponding expressions proposed by Eurocode 3 [3.23] are exactly the same and are written as:

$$\text{For} \quad n \leqslant a\text{:} \qquad M_{N,z,Rd} = M_{pl,z,Rd} \tag{3.83a}$$

$$\text{For} \quad n > a\text{:} \qquad M_{N,z,Rd} = M_{pl,z,Rd}\left[1 - \left(\frac{n - \mathrm{a}}{1 - \mathrm{a}}\right)^2\right] \tag{3.83b}$$

where n and a have the same significance as for eq. (3.81), i.e. $a = \alpha_w$.

Fig. 3.24 illustrates interaction diagrams representing eq. (3.83) for two profiles. It may be seen that the bending capacity is not reduced for small axial forces that may be resisted by the web alone, as expressed by equations (3.82a) or (3.83a). If this limit axial force is exceeded, the flanges start to participate in the axial resistance. Accordingly bending interacts with axial forces in the flanges that are of rectangular section and this interaction is governed by a parabolic curve as seen in Fig. 3.24 and equations (3.82b) or (3.83b).

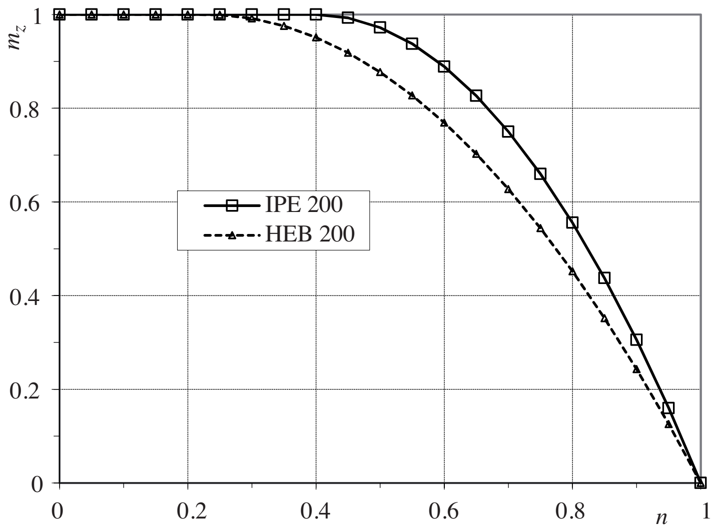


Fig. 3.24. Interaction diagrams to EN 1993-1-1 [3.23] for weak axis bending with axial force

3.8.2.4 Combination N, M_y, M_z

For the general case eq. (3.79) applies. The corresponding expressions proposed by Eurocode 3 [3.23] for *I* or *H* sections are written as following:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leqslant 1 \tag{3.84}$$

where:

$M_{N,y,Rd}$ is given by eq. (3.81)
$M_{N,z,Rd}$ is given by eq. (3.83) and
$\alpha = 2$; $\beta = 5n$ but $\beta \geqslant 1$.

Fig. 3.25a illustrates interaction diagrams for rolled profiles in accordance to the Eurocode 3 provisions, eq. (3.84), and the general formula, eq. (3.79). It may be seen that Eurocode 3 provides values close to, but not coincident to the general,

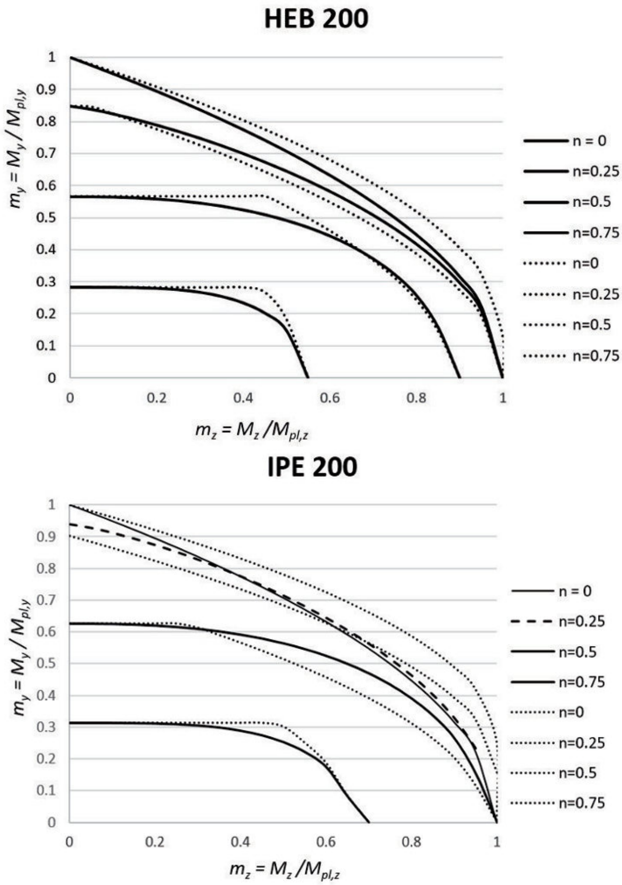


Fig. 3.25. Interaction diagrams to EN 1993-1-1 (continuous) and eq. (3.79) (dashed)

exact, solution. It may also be seen that Eurocode provides in some area for the IPE-profile a *lower* interaction curve when the axial force is zero $(n = 0)$ compared to the curve for the presence of an axial force $n = 0.25$. This indicates that the existing interaction relation of Eurocode 3 should be reconsidered.

3.8.3 Combination $N - M_y - M_z$ for hollow sections

Plastic interaction relationships for hollow sections were derived in [3.18], [3.25], [3.26]. In the following the procedures developed in [3.25] for SHS or RHS profiles will be presented and compared with the Eurocode 3 provisions [3.23]. For the relevant American provisions reference is made to [3.24].

In [3.25] the profile is represented by the centroid axes of their walls as in Fig. 3.26. Like for I-sections, this idealization is accurate when the lengths of the cross-section walls are sufficiently large compared with the correspondent wall thicknesses. Here again the influence of rounded corners is neglected.

The stress distribution in the cross-section at the full plastic state is illustrated in Fig. 3.26.

The design internal forces and moments $N_{Ed}, M_{y,Ed}, M_{z,Ed}$ may be represented in non-dimensional form by division with the corresponding design plastic resistances, where the design forces and moments with the index Ed are considered with absolute, positive, values:

$$n = N_{Ed}/N_{pl,Rd} \tag{3.85a}$$

$$m_y = M_{y,Ed}/M_{pl,y,Rd} \tag{3.85b}$$

$$m_z = M_{z,Ed}/M_{pl,z,Rd} \tag{3.85c}$$

The properties of this cross-section are similar to I-sections with following difference concerning the total area of the webs:

$$A_w = 2 \cdot h \cdot t_w \tag{3.85d}$$

Bending capacities for strong axis y-y:

Flanges: $$M_{pl,y,f} = \frac{\alpha_f}{2} h \cdot A \cdot f_y \tag{3.86a}$$

Webs: $$M_{pl,y,w} = \frac{2 \cdot h^2 \cdot t_w}{4} f_y = \frac{\alpha_w}{4} h \cdot A \cdot f_y \tag{3.86b}$$

Plastic section modulus: $$W_{pl,y} = \frac{1+\alpha_f}{4} \cdot h \cdot A \tag{3.86c}$$

Bending capacities for weak axis z-z:

Flanges: $$M_{pl,z,f} = \frac{2 \cdot b^2 \cdot t_f}{4} f_y = \frac{1-\alpha_w}{4} \cdot b \cdot A \cdot f_y \tag{3.87a}$$

Webs: $$M_{pl,z,w} = \frac{\alpha_w}{2} b \cdot A \cdot f_y = \frac{1-\alpha_f}{2} \cdot b \cdot A \cdot f_y \tag{3.87b}$$

Plastic section modulus: $$W_{pl,z} = \frac{1+\alpha_w}{4} \cdot b \cdot A \tag{3.87c}$$

In the above equations (3.86) and (3.87) the parameters α_f and α_w represent the contributions of the flanges and correspondingly the web to the total area as defined by equation (3.64a) and (3.64b).

The non-dimensional partial axial force capacities of the flanges and correspondingly the webs are written as:

$$n_f = \frac{N_{pl,f}}{N_{pl}} = \frac{N_{pl,f,Rd}}{N_{pl,Rd}} = \alpha_f \tag{3.88a}$$

$$n_w = \frac{N_{pl,w}}{N_{pl}} = \frac{N_{pl,w,Rd}}{N_{pl,Rd}} = \alpha_w = 1 - \alpha_f \tag{3.88b}$$

Obviously, for square hollow sections (SHS) with uniform thickness it is $\alpha_f = \alpha_w = 0.5$.

The non-dimensional partial bending moment capacities for the flanges and the webs are:

flanges:
$$m_{y,f} = \frac{M_{pl,y,f}}{M_{pl,y,}} = \frac{M_{pl,y,f,Rd}}{M_{pl,y,Rd}} = \frac{2\cdot\alpha_f}{1+\alpha_f} \tag{3.89a}$$

$$m_{z,f} = \frac{M_{z,fp}}{M_{z,p}} = \frac{M_{z,fpl,Rd}}{M_{z,pl,Rd}} = \frac{\alpha_f}{1+\alpha_w} = \frac{1-\alpha_w}{1+\alpha_w} \tag{3.89b}$$

webs:
$$m_{y,w} = \frac{M_{pl,y,w}}{M_{pl,y}} = \frac{M_{pl,y,w,Rd}}{M_{pl,y,Rd}} = \frac{\alpha_w}{1+\alpha_f} = \frac{1-\alpha_f}{1+\alpha_f} \tag{3.89c}$$

$$m_{z,w} = \frac{M_{pl,z,w}}{M_{pl,z}} = \frac{M_{pl,z,w,Rd}}{M_{pl,z,,Rd}} = \frac{2\cdot\alpha_w}{1+\alpha_w} \tag{3.89d}$$

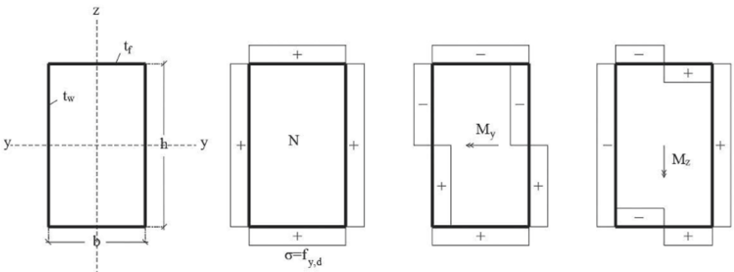


Fig. 3.26. Geometric idealization and stress distribution at plastic states for RHS

For the interaction relationship of this type of cross-section four cases shall be considered. In the following the derivation for only one case will be presented. For the complete derivation reference is made to the corresponding literature.

For the case analyzed here it is assumed that the strong axis design moments $M_{y,Ed}$ divided by the bending capacities of the relevant flange are larger than the corresponding values for weak axis bending so that following relations hold:

$$\frac{M_{y,Ed}}{M_{pl,y,f}} \geqslant \frac{M_{z,Ed}}{M_{pl,z,f}} \tag{3.90a}$$

or in non-dimensional form:

$$\frac{m_y \cdot (1+\alpha_f)}{2 \cdot \alpha_f} \geqslant \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w} \tag{3.90b}$$

For this case the axial force N and the bending moment M_z are resisted by the webs. The portions of the webs that resist the axial force are equal to, Fig. 3.27:

$$\lambda_N = \frac{N_{Ed}}{N_{pl,w}} = \frac{N_{Ed}}{N_{pl}} \cdot \frac{N_{pl}}{N_{pl,w}} = \frac{n}{\alpha_w} \tag{3.91a}$$

The portions of the webs that resist M_z are equal to:

$$\lambda_z = \frac{M_{z,Ed}}{M_{pl,z,w}} = \frac{M_{z,Ed}}{M_{pl,z}} \cdot \frac{M_{pl,z}}{M_{pl,z,w}} = = \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w} \tag{3.91b}$$

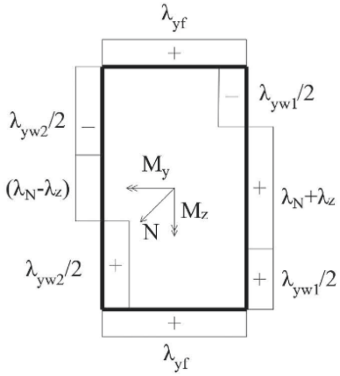


Fig. 3.27. Stress distribution in the flanges and the webs for hollow sections in which $\frac{M_{y,Ed}}{M_{pl,y,f}} \geqslant \frac{M_{z,Ed}}{M_{pl,z,f}}$

The sign of the stresses on the webs is the same due to N but different due to M_z. Accordingly, one web is stressed more than the other but the capacity of none is fully exploited so that it is:

$$\lambda_N + \lambda_z \leqslant 1 \quad \text{or} \quad \frac{n}{\alpha_w} + \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w} \leqslant 1 \tag{3.92}$$

Accordingly, both webs are able to participate in the resistance to M_y, however one more than the other due to the fact that they already have been exploited, one in the portion $\lambda_N + \lambda_z$ and the other in the portion $|\lambda_N - \lambda_z|$. For each web, N and M_z are resisted as with stresses of the same sign as axial forces while M_y with stresses of opposite sign as bending moments. For each web regarded as a rectangular cross-section the interaction $m + n^2 = 1$ holds, so that the portions to resist moments M_y are determined as following, Fig. 3.27:

Right web $\quad n = \lambda_N + \lambda_z, \quad m = \lambda_{yw1} \rightarrow \lambda_{yw1} = 1 - (\lambda_N + \lambda_z)^2 \quad$ (3.93a)

Left web $\quad n = |\lambda_N - \lambda_z|, \quad m = \lambda_{yw1} \rightarrow \lambda_{yw2} = 1 - (\lambda_N - \lambda_z)^2 \quad$ (3.93b)

Sum of the resisted moment:

$$\lambda_{yw} = \lambda_{yw1} + \lambda_{yw2} = 1 - (\lambda_N^2 + \lambda_z^2) \tag{3.93c}$$

The interaction relationship writes then:

$$\frac{m_y \cdot (1+\alpha_f) - \alpha_w + \{[0.5 \cdot m_z \cdot (1+\alpha_w)]^2 + n^2\}/\alpha_w}{2 \cdot \alpha_f} \leqslant 1 \tag{3.94}$$

For the other three cases similar relationships may be derived. The complete relations are summarized in Table 3.2.

Table 3.2. Interaction relationships for box sections

Case 1
Conditions $\frac{m_y \cdot (1+\alpha_f)}{2 \cdot \alpha_f} \geqslant \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w}$ and $\frac{n}{\alpha_w} + \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w} \leqslant 1$
Interaction relation(s) $\frac{m_y \cdot (1+\alpha_f) - \alpha_w + \{[0.5 \cdot m_z \cdot (1+\alpha_w)]^2 + n^2\}/\alpha_w^2}{2 \cdot \alpha_f} \leqslant 1$
Case 2
Conditions $\frac{m_y \cdot (1+\alpha_f)}{2 \cdot \alpha_f} < \frac{m_z \cdot (1+\alpha_w)}{2 \cdot \alpha_w}$ and $\frac{n}{\alpha_f} + \frac{m_y \cdot (1+\alpha_f)}{2 \cdot \alpha_f} \leqslant 1$
Interaction relation(s) $\frac{m_z \cdot (1+\alpha_w) - \alpha_f + \{[0.5 \cdot m_y \cdot (1+\alpha_f)]^2 + n^2\}/\alpha_f^2}{2 \cdot \alpha_w} \leqslant 1$
Case 3
Conditions $m_y \geqslant \frac{\alpha_w^2/4 - 4 \cdot (1-n)^2 - 4 \cdot \alpha_w \cdot n + 4 \cdot \alpha_w \alpha_f}{2 \cdot \alpha_w \cdot (1+\alpha_f)}$ and $m_z \geqslant \frac{\alpha_f^2/4 - 4 \cdot (1-n)^2 - 4 \cdot \alpha_f \cdot n + 4 \cdot \alpha_w \cdot \alpha_f}{2 \cdot \alpha_f \cdot (1+\alpha_w)}$
Interaction relation(s) $m_y \leqslant \frac{1}{0.5 \cdot (1+\alpha_f)} \cdot \left[\frac{\alpha_w}{16} + \alpha_f - n\right]$ and $m_z \leqslant \frac{1}{0.5 \cdot (1+\alpha_w)} \cdot \left[\frac{\alpha_f}{16} + \alpha_w - n\right]$
Case 4
Conditions $m_y < \frac{\alpha_w^2/4 - 4 \cdot (1-n)^2 - 4 \cdot \alpha_w \cdot n + 4 \cdot \alpha_w \cdot \alpha_f}{2 \cdot \alpha_w \cdot (1+\alpha_f)}$
Interaction relation(s) $m_z \leqslant \frac{1}{0.5 \cdot (1+\alpha_w)} \cdot \left[\frac{\alpha_f}{16} + \alpha_w - n\right]$
Case 5
Conditions $m_z < \frac{\alpha_f^2/4 - 4 \cdot (1-n)^2 - 4 \cdot \alpha_f \cdot n + 4 \cdot \alpha_w \cdot \alpha_f}{2 \cdot \alpha_f \cdot (1+\alpha_w)}$
Interaction relation(s) $m_y \leqslant \frac{1}{0.5 \cdot (1+\alpha_f)} \cdot \left[\frac{\alpha_w}{16} + \alpha_f - n\right]$

Eurocode 3 [3.23] proposes for RHS sections a simplified formula that leads to similar results to the accurate one presented above. This writes as following:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leqslant 1 \tag{3.95}$$

where:

$$M_{N,y,Rd} = M_{pl,y,Rd} \cdot \frac{1-n}{1-0.5\cdot\alpha_w} \quad \text{but} \quad \alpha_w \leqslant 0.5 \tag{3.96a}$$

$$M_{Nz} = M_{pl,z,Rd} \cdot \frac{1-n}{1-0.5\cdot\alpha_f} \quad \text{but} \quad \alpha_f \leqslant 0.5 \tag{3.96b}$$

$$\alpha = \beta = \frac{1.66}{1-1.13\cdot n^2} \leqslant 6 \tag{3.96c}$$

$$n = N_{Ed}/N_{pl,Rd}$$

– for rolled sections:	$a_w = (A-2bt)/A$	but	$a_w \leqslant 0.5$
	$a_f = (A-2ht)/A$	but	$a_f \leqslant 0.5$
– for welded sections:	$a_w = (A-2bt_f)/A$	but	$a_w \leqslant 0.5$
	$a_f = (A-2ht_w)/A$	but	$a_f \leqslant 0.5$

Interaction diagrams for two hollow sections with different α_f ratios are illustrated in Fig. 3.28. The comparison between the exact formulae [3.25] with the Eurocode 3 provisions [3.23] that the accuracy of the latter increases with decreasing axial force and are most accurate for profiles with equal flange and web areas, i.e. for SHS profiles with constant wall thickness.

3.8.4 Combination $N - M_y - M_z$ for circular hollow sections

This type of section is fully symmetric so there are no distinct principal axes and the resultant moment M_{Ed} is considered. According to the provisions of Eurocode 3 [3.23], the reduced moment capacity allowing for an axial force is equal to:

$$M_{N,Rd} = 1.04 \cdot M_{pl,Rd} \cdot (1-n^{1.7}) \tag{3.97}$$

where $n = N_{Ed}/N_{pl,Rd}$.

The design criterion is written as:

$$M_{Ed} \leqslant M_{N,Rd} \tag{3.98}$$

Interaction relationships for elliptical cross-sections, are given in [3.27], [3.28].

3.8.5 Combination $N - M_y - M_z$ for equal leg angle sections

Unlike American Codes [3.29], Eurocode 3 [3.23] does not include provisions for the plastic design of angle sections subjected to combined loading. In the following plastic interaction relations for angle sections under simultaneous internal forces and moments derived in [3.30] will be presented.

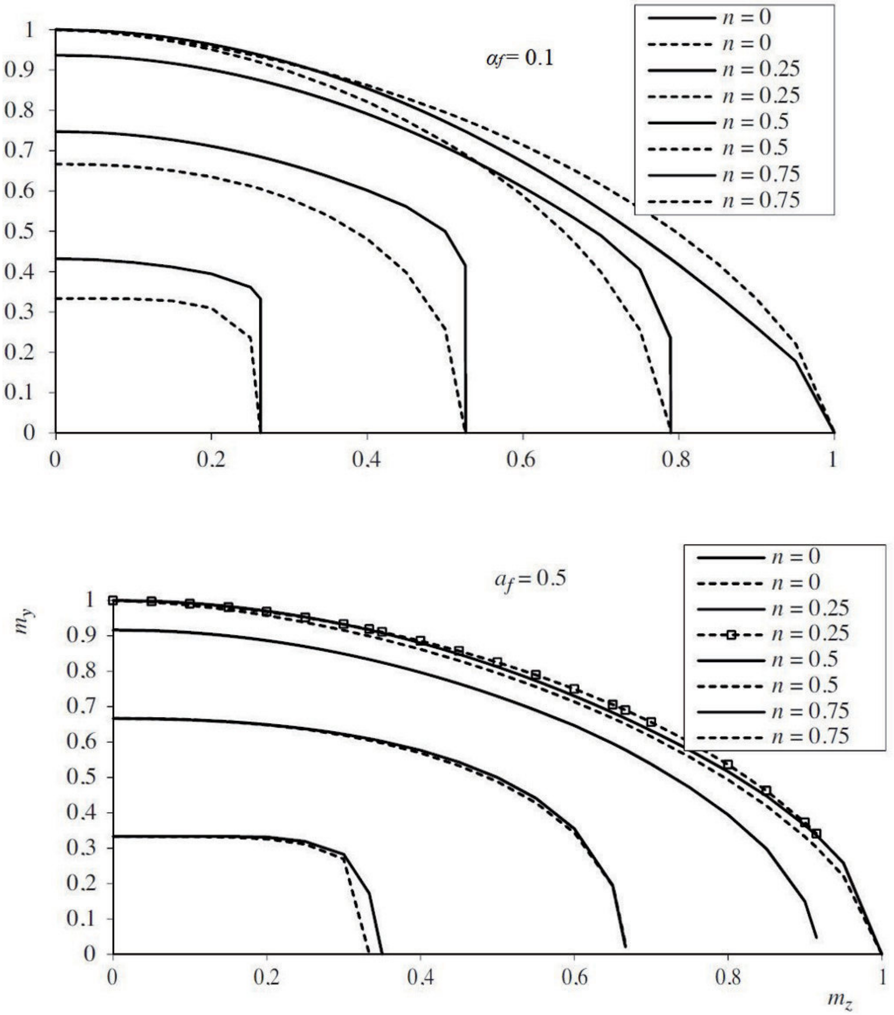


Fig. 3.28. Interaction diagrams for hollow sections according to EN 1993-1-1 (dashed) and as derived in [3.25] (continuous)

Angle sections may be represented by the centroid axes of their walls as in Fig. 3.29. This idealization is approximate for rolled profiles since it neglects the existence of the rounded corner at the junction of the legs. The stress distributions in the cross-section for axial forces and principal axes bending moments at the full plastic state are illustrated in Fig. 3.29. The design plastic resistances are equal to:

$$N_{pl,Rd} = A \cdot f_{y,d} \tag{3.99}$$

$$M_{pl,u,Rd} = W_{pl,u} \cdot f_{y,d} \tag{3.100}$$

$$M_{pl,v,Rd} = W_{pl,v} \cdot f_{y,d} \tag{3.101}$$

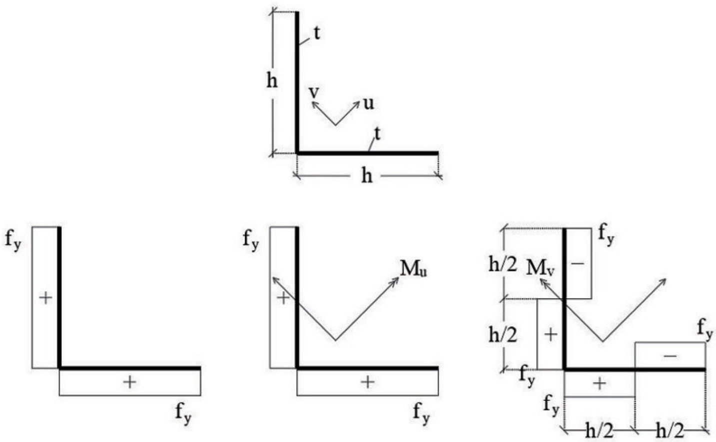


Fig. 3.29. Geometric idealization and stress distribution at plastic states for angle sections

The design internal forces and moments in non-dimensional form are considered with absolute, positive, values and are written as:

$$n = N_{Ed}/N_{pl,Rd} \tag{3.102}$$

$$m_u = M_{u,Ed}/M_{pl,u,Rd} \tag{3.103}$$

$$m_v = M_{v,Ed}/M_{pl,v,Rd} \tag{3.104}$$

The stress distribution and the stress resultants in each leg for combined axial force and weak axis bending moment are given in Fig. 3.30. The axial force is resisted by stresses of equal sign around the leg middle axis in a width equal to $n \cdot h$, while the moment by stresses of opposite sign in the remaining area. This distribution leads to following plastic interaction relationship:

$$n^2 + m_v = 1 \tag{3.105}$$

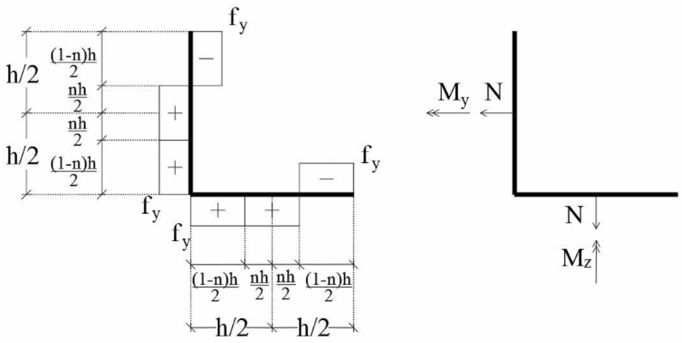


Fig. 3.30. Stresses and stress resultants in legs due to $N + M_v$

The stress distribution and the stress resultants in each leg for combined strong and weak axis moments are given in Fig. 3.31. The moment M_u is resisted by stresses of equal sign, but different from leg to leg, around the leg middle axis in a width equal to $m_u \cdot h$, while the moment M_v by stresses of opposite sign in the remaining area. This stress distribution leads to following plastic interaction relationship:

$$m_u^2 + m_v = 1 \tag{3.106}$$

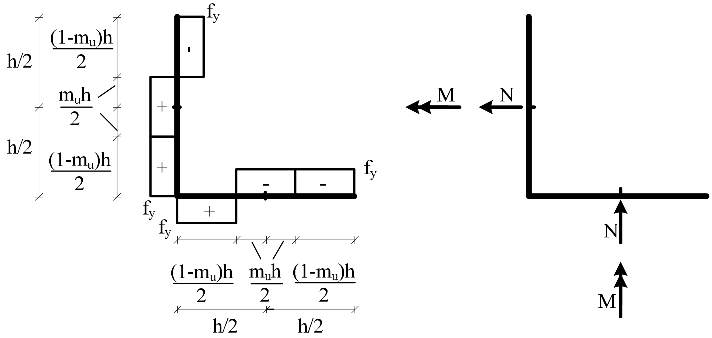


Fig. 3.31. Stresses and stress resultants in legs due to $M_u + M_v$

The combination of equations (3.105) and (3.106) provides for biaxial bending with axial force following simple interaction formula where all forces and moments are considered with positive values:

$$(|n| + |m_u|)^2 + |m_v| = 1 \tag{3.107}$$

Fig. 3.32 shows an interaction diagram based on this formula. For more refined analysis reference is made to the literature.

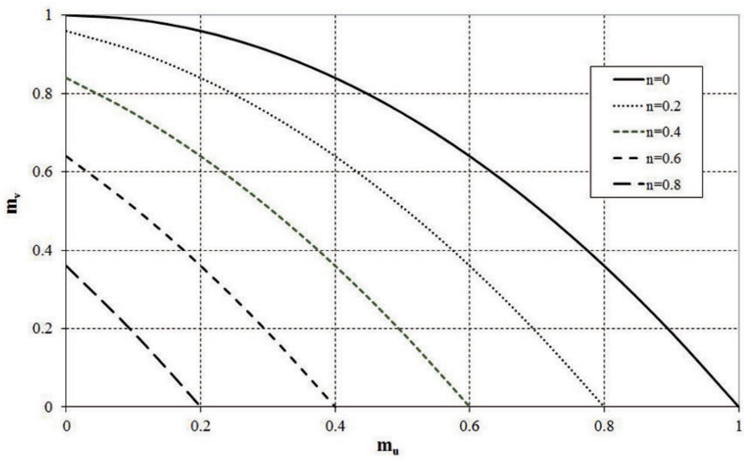


Fig. 3.32. Inelastic interaction diagram for equal leg angles

3.8.6 Linear interaction for all types of cross-sections

A conservative approach, applicable for all types of cross-sections, may be defined by following linear interaction formula:

$$\frac{N_{Ed}}{N_{pl,Rd}} + \frac{M_{y,Ed}}{M_{pl,y,Rd}} + \frac{M_{z,Ed}}{M_{pl,z,Rd}} \leqslant 1 \tag{3.108}$$

where:

N_{Ed}, is the design axial force
$M_{y,Ed}$ and $M_{z,Ed}$ are the design moments along the strong and weak principal axes whereas
$N_{pl,Rd}$, $M_{pl,y,Rd}$ and $M_{pl,z,Rd}$ are the corresponding plastic design resistances.

3.8.7 Influence of shear forces

For shear forces V_{Ed} higher than those prescribed by eq. (3.51), part of the material strength is exploited to resist shear. The relevant walls are then not able to develop the full yield strength to resist bending moments (or axial forces). Accordingly, the bending resistance may be determined for a cross-section with the same geometry, but with reduced yield strength of those walls that resist high shear forces, Fig. 3.33. The reduced yield strength may be determined from:

$$f_{y,red} = (1-\rho)\cdot f_{yd} \tag{3.109}$$

where:

$$\rho = \left(\frac{2\cdot V_{Ed}}{V_{Rd}} - 1\right)^2 \tag{3.110}$$

$f_{yd} = f_y/\gamma_{M0}$
V_{Ed} = the design shear force of the relevant wall resulting in from vertical shear and torsion, Fig. 3.34.
V_{Rd} = the corresponding design shear resistance of the wall.

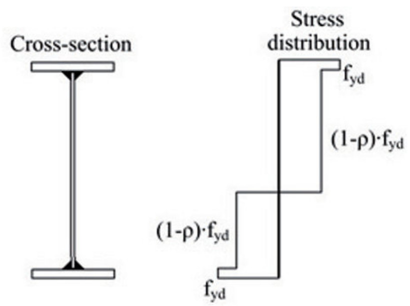


Fig. 3.33. Reduced yield strength for walls resisting high shear forces

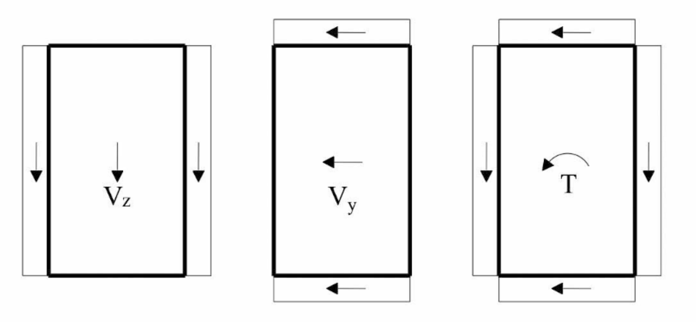


Fig. 3.34. Shear forces in walls of hollow sections due to shear and torsion

The design shear forces of hollow section walls result in from the sum of the forces due to vertical or horizontal shear and uniform torsion, Fig. 3.34. For open sections where torsion is resisted by warping the shear forces in the walls due to torsion result in from the sum of the secondary shear warping stresses within this wall, Fig. 3.16.

For open sections torsion is resisted mainly by warping and secondary shear forces develop in the flanges that sum up with those due to horizontal shear forces, Fig. 3.35.

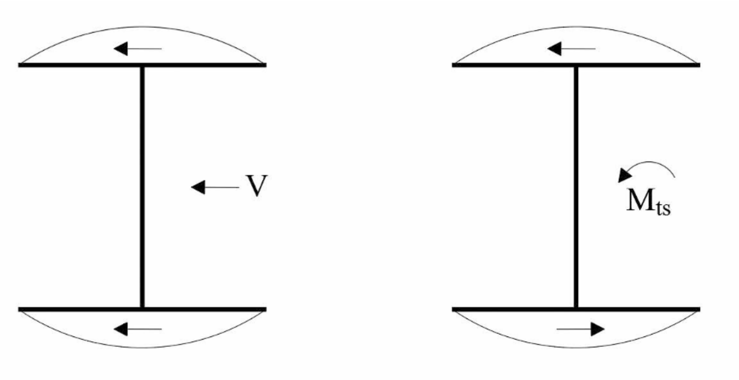


Fig. 3.35. Shear forces in walls of open sections due to shear and secondary warping torsion

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4
Member design

Abstract. This chapter describes methods for checking structural stability, such as flexural, torsional, lateral torsional or local buckling of members or cross-section walls. It gives the procedures to define the design buckling resistance, which are, according to Eurocode 3, similar for all types of instability. The evaluation proceeds on four steps: a) determination of the critical elastic, Euler, load, b) calculation of the relative slenderness, c) evaluation of the reduction factor to buckling and d) determination of the buckling resistance by application of this factor to the yield load with due consideration of safety. Useful information is given at each step, for example for the Euler load which is calculated by differential equations or the energy method. In addition, recommendations and modelling possibilities for design by means of numerical non-linear analysis methods, as well as ways for the application of the very promising general method as defined by Eurocode 3 are given. The chapter ends with design methods for plate girders composed of walls susceptible to local buckling, with guidance for design of laced or battened built-up members and with verification procedures for composite girders consisting of steel beams and concrete flanges.

4.1 General

Cross-section design alone may not be sufficient to check frame stability and should be complemented by member design. While a cross-section is not associated with a certain length, a member is a physical element of a certain length with constant or variable cross-section. Such an element, when completely or partly in compression or shear, may be subject to stability phenomena that cannot be checked without consideration of the type of support along its length. For example, columns subjected to compression may buckle between fixed supports. Similarly, the compression flanges of beams under bending may displace laterally between fixed lateral supports so that the beam as a member is subject to lateral torsional buckling. Plate girders are also subject to local or shear buckling and shall be checked for stability. For these reasons member design is required for checking stability, in addition to cross-section design that checks strength. An alternative way to check stability is to perform geometric non-linear analysis with due consideration of imperfections, in which case cross-section design is sufficient.

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As stated before, stability problems appear in different forms such as:

- Flexural buckling, where the member is subjected to transverse displacements,
- Torsional buckling, where the member is subjected to rotations,
- Lateral torsional buckling, where the member is subjected to rotations and lateral displacements of its compression flange,
- Local buckling or shear buckling, where the cross-section walls are subjected to deformations out-of-plane of the wall.

The limit stress for stability is the ratio between the basic strength and the partial safety factor. The basic strength is:

- For direct stresses, the yield strength f_y and
- For shear stresses, the shear strength.

The partial safety factor for stability is γ_{M1}, with recommended values for buildings $\gamma_{M1} = 1.0$ and for bridges $\gamma_{M1} = 1.10$ [4.1] to [4.3].

4.2 Flexural buckling of compression members

4.2.1 Elastic critical (Euler) loads

Critical buckling loads of compression members may be determined under the assumptions that [4.4]:

- The axial force is concentric with no eccentricity.
- The member is absolutely straight.
- The material behavior is elastic throughout loading.

Member stability may be examined by consideration of the equilibrium of an infinitesimal element in the deformed state, which for the axially loaded member leads to following differential equation, Figure 4.1:

$$E \cdot I \cdot w'''' + N \cdot w'' = 0 \quad (4.1)$$

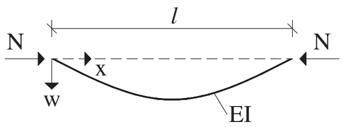


Fig. 4.1. Compression member

The solution of this homogeneous equation is:

$$w = a_1 \cdot \sin \rho\xi + a_2 \cdot \cos \rho\xi + a_3 \cdot \rho \cdot \xi + a_4 \quad (4.2)$$

where:

$$\xi = x/l$$

and ρ a parameter called “strut index” determined from:

$$\rho = l \cdot \sqrt{N/EI} \quad (4.3)$$

The parameters a_i may be determined from the support conditions, which for the simply supported member of Figure 4.1 are as following:

$$w(0) = w(l) = 0$$
$$M(0) = M(l) = 0 \quad \text{or} \quad w''(0) = w''(l) = 0$$

The three first conditions give $a_2 = a_3 = a_4 = 0$ and the last becomes then:

$$w''(l) = -a_1 \cdot \rho^2 \cdot \sin \rho l = 0$$

The non-trivial solution provides the buckling condition $\sin \rho l = 0$, which gives:

$$\rho = \frac{i \cdot \pi}{l} \qquad i = 1, 2, 3, \ldots \tag{4.4}$$

By substitution in (4.2) the deflections may be determined as following:

$$w = a_1 \cdot \sin i\pi\xi \qquad i = 1, 2, 3, \ldots \tag{4.5}$$

where i represents the number of the buckling mode.

The displacements w for the first four buckling modes are illustrated in Figure 4.2. This analysis provides only the form of the buckling modes but no values for the displacements, due to the fact that the parameter a_1 is undefined. However, it allows the determination of the buckling loads corresponding to each buckling mode. Indeed, introducing ρ on the left side of (4.4) the critical loads for the buckling modes i are determined from:

$$N_{cr,i} = i^2 \cdot \frac{\pi^2 \cdot EI}{l^2} \tag{4.6}$$

Evidently the first buckling mode provides the smallest critical load that is equal to:

$$N_{cr} = \frac{\pi^2 \cdot EI}{l^2} \tag{4.7}$$

This load is also called Euler load from the name of the Swiss Engineer that solved this problem first in 1777. For support conditions other than simple support, the Euler

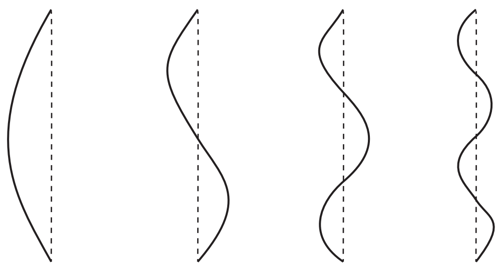


Fig. 4.2. Buckling modes for a simply supported compression column

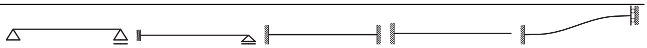
load is defined by the more general expression by introduction of the critical buckling length l_{cr}:

$$N_{cr} = \frac{\pi^2 \cdot EI}{(\beta \cdot l)^2} = \frac{\pi^2 \cdot EI}{l_{cr}^2} \tag{4.8}$$

where:

l is the actual length of the member
l_{cr} is the buckling length and
β is the buckling length coefficient as a function of the support conditions, Table 4.1.

Table 4.1. Buckling length coefficients β for various support conditions



Support conditions	SS SS	C SS	C C	cantilever	C C Sway ends
Buckling length coefficient $\beta = l_{cr}/l$	1	0.7	0.5	2	1
Notation	SS simply supported C fixed				

The critical buckling stress is determined by division of the critical load by the cross-section area and is found from:

$$\sigma_{cr} = \frac{\pi^2 \cdot E}{\lambda^2} \tag{4.9}$$

where:

$$\lambda = \frac{l_{cr}}{i} = \text{member slenderness} \tag{4.10}$$

$$i = \sqrt{\frac{I}{A}} = \text{radius of gyration of the cross-section}$$

Equation (4.9) defines a hyperbola, called the Euler hyperbola. It may be seen that the critical stresses increase quickly for small slenderness as a consequence of the assumed elastic behavior and may exceed the yield strength. This is a first indication that the Euler theory provides ideal, but not ultimate limit loads. However, it will be seen later that this theory serves as a basis to define ultimate limit loads to be used in engineering design.

Alternatively, critical buckling loads may be determined by the energy method which examines the system's potential in an initial un-deformed equilibrium state O and a deformed one in its neighborhood I, Figure 4.3. The total potential at the equilibrium states O and I is stationary so that its variation is zero, $\delta V_O = \delta V_I = 0$. Considering that potential at state I results from the potential at state O plus a differential potential ΔV and expanding it in a Taylor series it is:

$$\delta V_I = \delta(V_O + \Delta V_O) = \delta\left(V_O + \delta V_O + \frac{1}{2!}\delta^2 V_O + \frac{1}{3!}\delta^3 V_O + \dots\right) = 0 \tag{4.11a}$$

Considering the zero terms at equilibrium states, the relation that defines the energy criterion for linear stability theory, or linear buckling analysis, may be derived:

$$\delta(\delta^2 V_O) = 0 \tag{4.11b}$$

Linear buckling theory defines only equilibrium states. Non-linear theory considering higher order terms and its derivatives must be employed to determine the type of equilibrium state. In fact, the equilibrium is neutral, unstable or stable, depending on whether the sign of $\delta^3 V_O$ is zero, negative or positive, Figure 4.3.

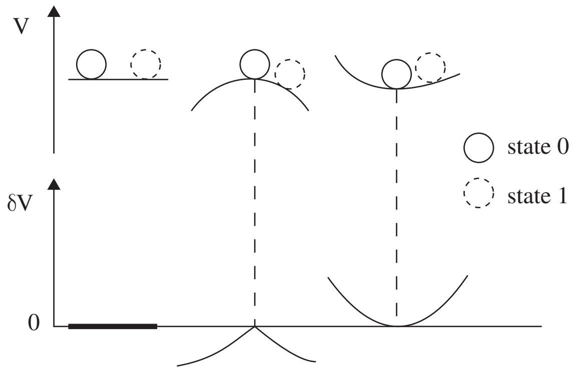


Fig. 4.3. Total potential of the system and its first derivative

The application of the energy method for the compression member of Figure 4.1 leads to following expression for the total potential of the system, where the first term expresses the internal energy of the system and the second term the work done by the applied force:

$$\delta^2 V = \int \left(\frac{1}{2} \cdot E \cdot I \cdot w''^2 + N \cdot \frac{1}{2} \cdot w'^2 \right) \cdot dx \tag{4.12}$$

Equation (4.11b) states that in order to define the critical load, the first variation, or first derivative, of the above expression must be set to zero. Indeed, assuming the shape for the first buckling mode to be described by equation $w = a \cdot \sin \pi\xi$, inserting it in (4.12), performing the integration and subsequently the differentiation, the critical load exactly as given by eq. (4.7) may be calculated.

In engineering practice critical buckling loads are determined numerically by means of LBA, see section 2.8.

4.2.2 Design buckling resistance

The assumptions on which linear buckling analysis is based are not valid in real steel members. Indeed, members in real steel structures are not absolutely straight but have geometrical imperfections, steel is not elastic but yields after a certain level of stress and loading is not absolutely concentric. In addition, residual stresses develop in

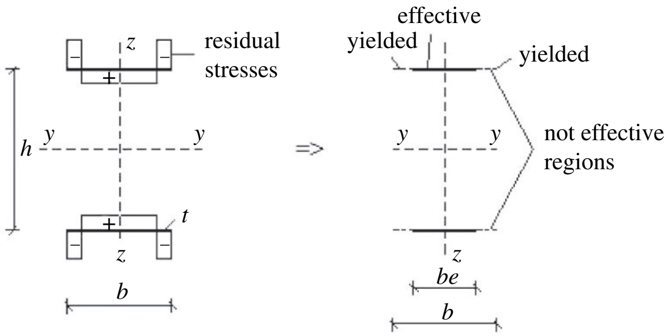


Fig. 4.4. Residual stresses and effective regions for the flanges of an I-section

steel members during the fabrication processes that lead to structural imperfections. All these effects have a negative influence on the compression capacity and generally the stability of actual members and need to be considered in design.

Taking as an example material yielding, it becomes evident that for small slenderness the limit stress cannot be defined by the critical stress of eq. (4.9) but by the yield stress, when the latter is smaller than the former.

In addition, residual stresses do not influence equally the buckling resistance. Assuming a simplified distribution of residual stresses for the flanges of an I-cross-section, Figure 4.4, it may be seen that by application of a compression load the edges yield first and at further loading become ineffective. Consequently, near the critical loading only the effective parts contribute to the bending stiffness (EI). The effective stiffness is different for the two principal axes according to:

$$EI_{y,\text{eff}} = b_e \cdot t \cdot h^2/4 \qquad EI_{z,\text{eff}} = b_e^3 \cdot t/6 \tag{4.13}$$

Comparing the ratios between the initial stiffness and the stiffness near the critical loading as expressed by equations (4.14), it may be seen that the stiffness reduction is not the same for the two principal axes. Indeed, the reduction in stiffness for the strong axis is linear, while for the weak axis in the 3$^{\text{rd}}$ power. Accordingly, weak axis buckling is more influenced by structural imperfections, i.e. residual stresses, than does strong axis buckling.

$$\frac{EI_{y,\text{eff}}}{EI_y} = \frac{b_e}{b}(<1) \qquad \frac{EI_{z,\text{eff}}}{EI_z} = \left(\frac{b_e}{b}\right)^3 \tag{4.14}$$

Design buckling resistances in Eurocode 3 are based on the application of the very old Ayrton-Perry formula, introduced in the 19$^{\text{th}}$ century [4.5]. This approach examines a compression member with equivalent geometrical imperfections, calculates the internal forces and moment by geometrically non-linear 2$^{\text{nd}}$ order analysis and makes a cross-section check at the most stressed mid-span section, Figure 4.5. Considering a linear interaction relationship, the design criterion may be written as:

$$\frac{N}{N_R} + \frac{N \cdot e_0}{M_R} \cdot \frac{1}{1 - \frac{N}{N_{cr}}} = 1 \tag{4.15}$$

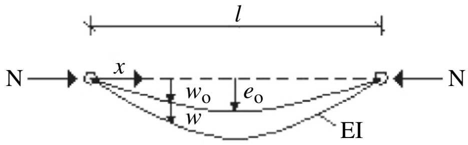


Fig. 4.5. Compression member having an initial geometrical imperfection

where

N is the applied axial force and
N_R and M_R are the cross-section resistances to compression and bending respectively.

The method searches for a reduction factor χ that applies to the axial resistance N_R $(= A \cdot f_y)$. Accordingly, it is:

$$\chi = \frac{N}{N_R} \tag{4.16}$$

The member slenderness may be written from eq. (4.9) as:

$$\lambda = \sqrt{\frac{\pi^2 \cdot E}{\sigma_{cr}}} \tag{4.17a}$$

and a reference slenderness analogously as:

$$\lambda_1 = \sqrt{\frac{\pi^2 \cdot E}{f_y}} \tag{4.17b}$$

so that a relative slenderness is introduced:

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{N_R}{N_{cr}}} \tag{4.18a}$$

or

$$\frac{N_r}{N_{cr}} = \bar{\lambda}^2 \tag{4.18b}$$

The initial imperfection may be written in the form:

$$e_0 = \frac{M_R}{N_R} \cdot \eta \tag{4.19}$$

where according to Robertson [4.6]

$$\eta = \alpha \cdot (\bar{\lambda} - 0.2) \tag{4.20}$$

where α is an imperfection factor.

Introducing (4.16) to (4.20) in (4.15), the reduction factor χ may be determined by solving following equation:

$$\chi + \chi \cdot \eta \cdot \frac{1}{1 - \chi \cdot \bar{\lambda}^2} = 1 \tag{4.21a}$$

or

$$\chi^2 + x \cdot \left(-1 - \frac{\eta}{\bar{\lambda}^2} - \frac{1}{\bar{\lambda}^2} \right) + \frac{1}{\bar{\lambda}^2} = 0 \tag{4.21b}$$

The solution of the above equation gives the reduction factor:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \tag{4.22a}$$

where:

$$\Phi = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2) \tag{4.22b}$$

The buckling resistance of compression members is then obtained from:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \tag{4.23}$$

where:

A is the gross cross-section area for class 1, 2 and 3 cross-sections
$A = A_{\text{eff}}$ is the effective area for class 4 cross-sections
$\chi = 1$ for $\bar{\lambda} \leq 0.2$.
χ is obtained from eq. (4.22) for $\bar{\lambda} > 0.2$.
$\gamma_{\text{M1}} = 1.0$ as recommended for buildings.

The design format is written as:

$$N_{Ed} \leqslant N_{b,Rd}$$

The reduction factor χ depends through eq. (4.22b) from the imperfection factor α. As a result of calibration with experimental and analytical/numerical investigations, five European buckling curves with different imperfection factors were established [4.7] to [4.9], as illustrated in Figure 4.6 and Table 4.2. These imperfections factors correspond to physical values of equivalent geometric bow imperfections that unite geometric and structural imperfections and are also shown in Table 4.2. Imperfections are different for various sections shapes and affect differently the buckling response about the strong and weak axis of the cross-section. Accordingly, the buckling curves are associated to shapes and dimensions of cross-sections and the axis about which buckling is considered as shown in Table 4.3. Following may be observed from Table 4.3:

- Hollow sections are associated with more beneficial buckling curves due to their higher resistance to torsion.
- Hot rolled sections are associated with more beneficial buckling curves compared to welded or cold formed sections due to their smaller structural imperfections.

Concluding, it may be seen how important the critical buckling load is. It does not give the member compression capacity but provides a reference value to determine a relative slenderness through eq. (4.18a) which is the input parameter for the calculation of the reduction factor and the design resistance.

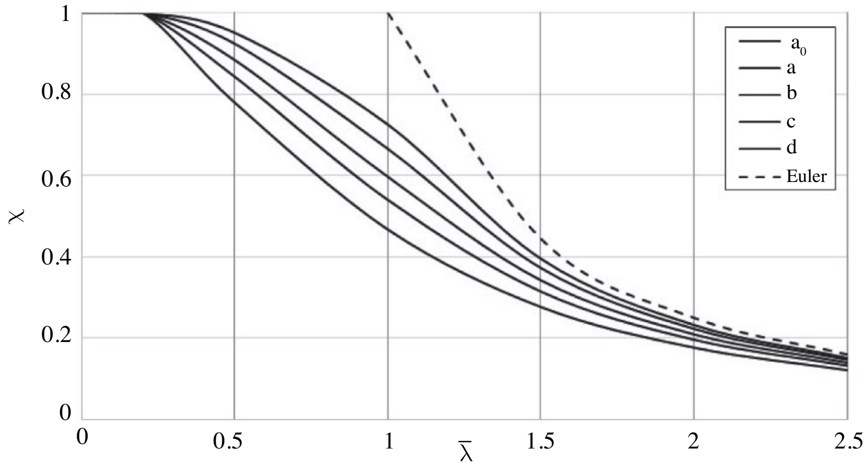


Fig. 4.6. European buckling curves and Euler hyperbola

Table 4.2. Imperfection factors α for European buckling curves [4.2]

Buckling curve	a_0	a	b	c	d
Imperfection factor α (α_{LT})	0.13	0.21	0.34	0.49	0.76
Bow imperfection e_0 for elastic design	l/350	l/300	l/250	l/200	l/150
Bow imperfection e_0 for plastic design	l/300	l/250	l/200	l/150	l/100

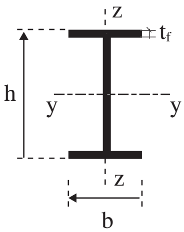
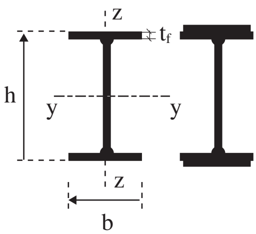
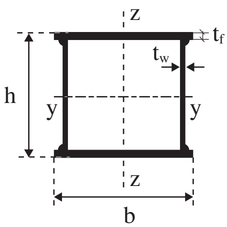
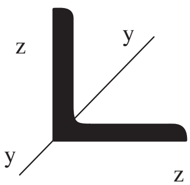
4.2.3 Design by non-linear analysis

The buckling resistance of compression members as defined in the previous section is well established. However, eq. (4.23) applies to members with constant cross-section. When the compression load or the cross-section varies along the member length (in case of tapered or non-uniform members, which are very often used in steel structures, e.g. in cranes, in portal or multi-storey frames), the buckling length coefficient β may be determined by numerical methods. Useful information by application of analytical methods may be found in references [4.48] to [4.56]. However, the question remains which cross-section area A should be introduced in eq. (4.23) to define the design compression resistance.

An alternative approach is the performance of geometrically non-linear analysis (2^{nd} order analysis) with due consideration of equivalent geometric imperfections. If analysis is elastic (GNIA), design is performed by cross-section checks along the member length. If analysis considers also materially non-linear effects (GMNIA), the equilibrium curve has a maximum value that corresponds to the design resistance. Figure 4.7 illustrates the analysis and design procedure. Geometrical bow imperfections should be introduced from Table 4.2 and be considered for one or the other principal axis of the cross-section but not simultaneously for both.

It should be said that non-linear analysis with due consideration of geometrical imperfections does not necessarily lead to the same result with the application of the

Table 4.3. Selection of buckling curves [4.2]

Cross-sections	*Limits*		*Buckling about axis*	*Buckling curve* $S235$ to $S420$	*Buckling curve* $S460$
Rolled sections 	$h/b > 1.2$	$t_f \leqslant 40\,\text{mm}$	*y-y*	*a*	a_0
			z-z	*b*	a_0
		$40\,\text{mm} < t_f \leqslant 100\,\text{mm}$	*y-y*	*b*	*a*
			z-z	*c*	*a*
	$h/b \leqslant 1.2$	$t_f \leqslant 100\,\text{mm}$	*y-y*	*b*	*a*
			z-z	*c*	*a*
		$t_f > 100\,\text{mm}$	*y-y*	*d*	*c*
			z-z	*d*	*c*
Welded I-sections 	$t_f \leqslant 40\,\text{mm}$		*y-y*	*b*	*b*
			z-z	*c*	*c*
	$t_f > 40\,\text{mm}$		*y-y*	*c*	*c*
			z-z	*d*	*d*
Hollow sections 	Hot finished		any	*a*	a_0
	Cold formed		any	*c*	*c*
Welded box sections 	Generally (except as below)		any	*b*	*b*
	Thick welds: $\alpha > 0.5 \cdot t_f$ $\frac{b}{t_f} < 30$, $\frac{h}{t_w} < 30$		any	*c*	*c*
L-sections 			any	*b*	*b*

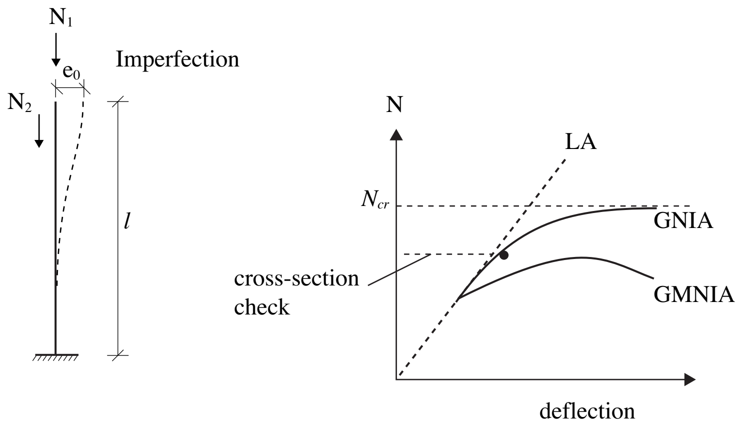


Fig. 4.7. Application of geometrically non-linear analysis

European buckling curves, even for simple compression members. One important reason is that imperfections for the European buckling rules were developed under the assumption of a linear $(M\text{-}N)$ interaction relationship for the cross-section, as defined by equation (4.15). However, as shown in chapter 3 the plastic interaction formulae for the various cross-sections are not linear. As a consequence, lots of investigations are currently in progress in order to define improved imperfection rules that lead to more consistent safety margins for many types of cross-sections and slenderness values, see [4.10] to [4.12].

4.2.4 Torsional and torsional-flexural buckling of compression members

Some types of open cross-sections, especially thin-walled, may be vulnerable to torsional buckling when subjected to axial compression. This is an instability mode where the member does not deflect but twists around its shear center. The elastic critical load may be determined from [4.4]:

$$N_{cr,T} = \frac{1}{i_M^2} \cdot \left(GI_t + \frac{\pi^2 \cdot EI_w}{l_T^2} \right) \tag{4.24}$$

where:

I_t is the torsion constant of the cross-section
I_w is the warping constant of the cross-section
$i_M^2 = i_y^2 + i_z^2 + y_M^2$ is the polar radius of inertia of the cross-section in respect to the shear center.
i_y, i_z are the radii of gyration of the cross-section about the strong and weak axis respectively
y_M is the distance between the centroid and the shear center of the cross-section in direction of the strong axis y
l_T is the buckling length in respect to torsion.

The elastic critical stress is determined by division with the cross-section area and is written as: $\sigma_{ct,T} = \frac{N_{cr,T}}{A}$.

The critical stress to torsional buckling is further reduced when torsional buckling interacts with flexural buckling to provide a combined torsional-flexural buckling mode. For mono-symmetric cross-sections, where *y-y* is the axis of symmetry (Table 4.3), the critical torsional-flexural buckling stress is determined from [4.13], [4.14]:

$$\sigma_{cr,TF} = \frac{1}{2 \cdot \beta} \cdot \left[(\sigma_{cr,y} + \sigma_{cr,T}) - \sqrt{(\sigma_{cr,y} + \sigma_{cr,T})^2 - 4 \cdot \beta \cdot \sigma_{cr,y} \cdot \sigma_{cr,T}} \right] \tag{4.25}$$

where:

$\sigma_{cr,y}$ is the critical stress for flexural buckling about the axis of symmetry y
$\sigma_{cr,T}$ is the critical stress for torsional buckling as above

$$\beta = 1 - \left(\frac{y_M}{i_p} \right)^2$$

y_M is as in equation (4.24)
$i_p^2 = i_y^2 + i_z^2$ is the polar radius of inertia of the cross-section.

This instability mode may be critical for very short member from rolled or welded cross-sections. Design for it follows the same procedure as for flexural buckling and is associated with buckling curve *b*.

4.3 Lateral torsional buckling (LTB) of bending members

4.3.1 Elastic critical moments

4.3.1.1 Unrestrained or discretely restrained members

Members under strong axis bending are subjected to displacements v in direction of the weak axis, Figure 4.8. Due to bending, their cross-sections are partly in compression and partly in tension. When equilibrium is considered in a displaced position it is found that the lateral displacements, v, of the two flanges differ. In the compression flange destabilizing forces appear, i.e. forces in direction of the assumed displacements. As a result, lateral displacements in the compression flange grow faster than in the tension flange, where stabilizing forces opposite to the assumed displacements develop. Due to the difference in lateral displacements the cross-section is twisting, mobilizing its torsional rigidity against the twist. The member is therefore subjected simultaneously to lateral displacements and torsion, this instability mode being called lateral torsional buckling [4.15], [4.16].

The critical state may be found by the same assumptions made for the Euler load: lack of imperfections and elastic behavior. The formulation of the equilibrium conditions in the deformed state leads to a system of three coupled differential equations for which, with very few exemptions, there is no analytical solution. Therefore,

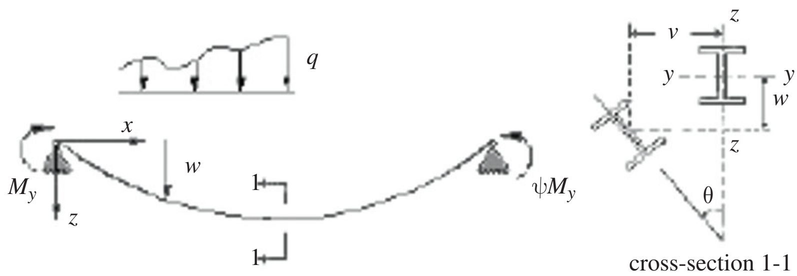


Fig. 4.8. Displacements and rotations of members under bending

problems of lateral torsional buckling are solved almost exclusively by the energy method. For an unrestrained beam of a doubly symmetric cross-section, Figure 4.9, the total potential may be expressed as a function of the angle of twist θ only and writes [4.17]:

$$\frac{1}{2}\bar{\delta}^2 V^T = \frac{1}{2}\cdot\int_0^l \left[EI_w\cdot(\theta'')^2 + GI_t\cdot(\theta')^2 - \frac{M_y^2}{EI_z}\cdot\theta^2 - q\cdot z_g\cdot\theta^2\right]\cdot dx - \\ -P\cdot z_g\cdot\theta_{(P)}^2 \tag{4.26a}$$

where:

GI_t = torsion rigidity (St Venant)
EI_w = warping rigidity
M_y = bending moments, strong axis
q = uniformly distributed loads, in direction of the weak axis
P = concentrated loads, in direction of the weak axis
θ = angle of twist
$\theta_{(P)}$ = angle of twist at the position of the concentrated loads
z_g = distance between load application point and shear center (= centroid) in direction of the weak axis.

In the above equation the first three terms express the internal elastic energy of the system. The first two terms define the elastic strain energy due to warping and torsion rigidity and the 3rd term the strain energy due to bending. The last two terms express the work done by the external forces. The beam is considered in the deformed state, so that the external loads become eccentric in respect to the shear center leading to torsion moments. Figure 4.9 shows that these moments may be destabilizing or stabilizing, depending on the position of load application, i.e. the sign of z_g.

As outlined before, the critical state is defined by the condition that the first derivative of the above expression becomes zero:

$$\delta\left(\frac{1}{2}\bar{\delta}^2 V^T\right) = \delta\left(\bar{\delta}^2 V^T\right) = 0 \tag{4.26b}$$

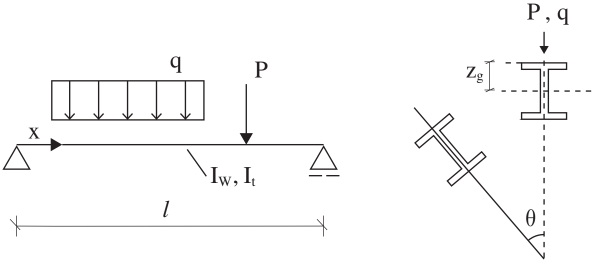


Fig. 4.9. Lateral torsional buckling of unrestrained beams

Numerical solutions may be found by application of the Ritz method, selecting one approximate shape function for θ that fulfills the boundary conditions. The parameters of this function are then determined by appropriate differentiation and integration of equations (4.26a) and (4.26b).

The application of this methodology is illustrated in the example of a simply supported I-beam subjected to uniform loading applied at the shear center/centroid as in Figure 4.10 [4.17]. The supports are simple also in respect to torsion, i.e. fork supports that restrain the angle of twist and resist torsion moments but do not restrain warping moments. The assumed shape function and its derivatives are as following:

$$\theta = C \cdot \sin\frac{\pi x}{l} = C \cdot \sin\pi\xi \tag{4.27a}$$

$$\theta' = C \cdot \frac{\pi}{l} \cdot \cos\frac{\pi x}{l} = C \cdot \frac{\pi}{l} \cdot \cos\pi\xi \tag{4.27b}$$

$$\theta'' = -C \cdot \left(\frac{\pi}{l}\right)^2 \cdot \sin\frac{\pi x}{l} = -C \cdot \left(\frac{\pi}{l}\right)^2 \cdot \sin\pi\xi \tag{4.27c}$$

where $\xi = x/l$.

It may be easily confirmed that this shape function with one parameter, C, is an acceptable function since it fulfills the conditions of the simple torsion supports, zero twist angle – zero warping moments: $\theta(0) = \theta(l) = 0$ and $\theta''(0) = \theta''(l) = 0$.

The bending moments along the beam and the critical, maximum, moment are given by:

$$M_y(\xi) = \frac{q \cdot l^2}{2} \cdot \left(\xi - \xi^2\right) = 4 \cdot M_{cr} \cdot \left(\xi - \xi^2\right) \tag{4.27d}$$

$$M_{cr} = \frac{q \cdot l^2}{8} \tag{4.27e}$$

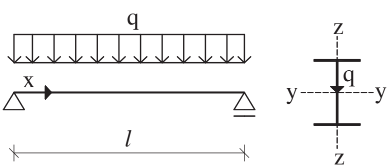


Fig. 4.10. Notation for an unrestrained I-beam subjected to uniform loading

Due to doubly symmetric cross-section and load application at the shear center it is $z_g = 0$, so that by introduction of (4.27) equation (4.26) is written as following:

$$\bar{\delta}^2 V^T = \int_0^1 C^2 \cdot \left\{ EI_w \cdot \left(\frac{\pi}{l}\right)^4 \cdot \sin^2 \pi\xi + GI_t \cdot \left(\frac{\pi}{l}\right)^2 \cos^2 \pi\xi - \right. \\ \left. - \frac{16 \cdot M_{cr}^2}{EI_z} \cdot \left(\xi - \xi^2\right)^2 \cdot \sin^2 \pi\xi \right\} \cdot d\xi \tag{4.27f}$$

Performing the differentiation (4.25) leads to following equilibrium condition:

$$2 \cdot C \cdot \left\{ \frac{EI_w}{2} \cdot \left(\frac{\pi}{l}\right)^4 + \frac{GI_t}{2} \cdot \left(\frac{\pi}{l}\right)^2 - \frac{16 \cdot M_{cr}^2}{41 \cdot EI_z} \right\} = 0 \tag{4.27g}$$

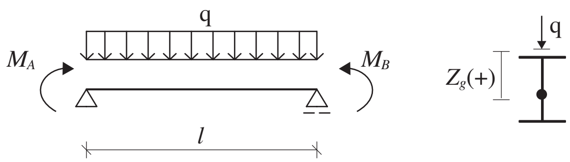
The non-trivial ($C \neq 0$) solution gives the critical lateral torsional buckling moment of the beam:

$$M_{cr} = 1.132 \cdot \frac{\pi^2 \cdot E \cdot I_z}{l^2} \cdot \sqrt{\frac{I_w}{I_z} + \frac{l^2}{\pi^2} \cdot \frac{GI_t}{EI_z}} \tag{4.28}$$

It may be seen that linear buckling analysis (LBA) as presented here provides the value of the critical moment by equation (4.28) and the shape of the buckling mode by equation (4.27a). However, it does not give values for the deformations since C is not defined.

For more general loading, critical moments for beams with doubly symmetric sections may be determined from Table 4.4. The critical moment refers to the end support B where the applied moment is higher, in absolute terms, compared to the

Table 4.4. Critical moments for unrestrained I-beams [4.17]

System	
Loading conditions	$-1 \leqslant \psi = \frac{M_A}{M_B} \leqslant 1, \quad 0 \leqslant \mu_0 = \frac{-M_B}{q \cdot l^2/8}$ M_A, M_B positive for direction of moment as shown
Critical moment	$M_{cr,B} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(kl)^2} \cdot \left[\sqrt{\frac{I_w}{I_z} + \frac{(kl)^2}{\pi^2} \cdot \frac{GI_t}{EI_z} + \left(C_2 \cdot z_g\right)^2} - \left(C_2 \cdot z_g\right) \right]$
Parameters	$C_1 = \frac{1}{\sqrt{2 \cdot I}}, k = 1, C_2 = \frac{0.28658}{\mu_0 \cdot \sqrt{I}}, I = \frac{1}{7} + \frac{\psi}{4.6} + \frac{\psi^2}{7} - \frac{1+\psi}{2.3\mu_0} + \frac{0.39}{\mu_0^2}$
Remarks	a) For a simply supported beam without end moments ($M_A = M_B = 0$), it may be set $\psi = 1$ and $\mu_0 = 1/1000$ b) For a simply supported beam without transverse loading ($q = 0$), μ_0 may be set equal to 100.

moment at the other support A. For other cases reference is made to the literature [4.2], [4.4], [4.13], [4.16] to [4.19].

From the expressions of the critical LTB moment, it may be seen that this instability mode refers mainly to members with open sections. Indeed, hollow sections are less susceptible to LTB due to their high torsional rigidity, GI_t, and consequently due to high critical moments and low slenderness.

4.3.1.2 Continuously restrained beams

In industrial buildings, purlins and side rails are connected to roof and wall elements such as trapezoidal sheeting or sandwich panels. Similarly, in floor decking, where the floor beams are compositely connected to the concrete slab through stud connectors. In such cases the connected flange is continuously restrained in respect to lateral displacements and/or rotations, Figure 4.11, [4.21] to [4.23].

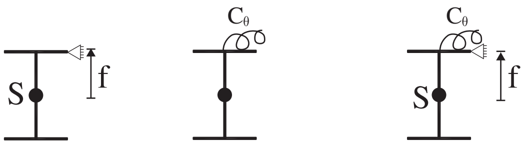


Fig. 4.11. Beams with continuous restraint of lateral displacements, rotations or both

If the compression flange is throughout laterally supported there is no risk of lateral torsional buckling. That is always the case for simply supported floor beams. However, LTB instability is possible if compression flange is the unconnected flange, such as for purlins or side rails subjected to negative wind pressure or for continuous beams. Analytical expressions based on Ritz analysis methods may be derived by considering additional terms, as $(c_\theta \cdot \theta^2)$ for the torsional restraint, in the expression for the total potential.

Critical loads for beams with continuous restraint of lateral displacements as determined by the authors are given in Table 4.5. For additional information reference is made in the literature [4.24].

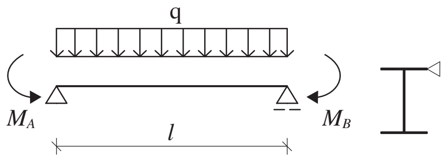
For rotation restraint the torsional spring stiffness c_θ is to be calculated under consideration of the flexibility due to deformations of the decking, distortion of the beam profile and of the connections between the flange and the sheathing, if applicable Figure 4.12. Since the deformations add the equivalent springs are in series and the resulting flexibilities add so that it is [4.13]:

$$\frac{1}{c_\theta} = \frac{1}{c_{\theta,de}} + \frac{1}{c_{\theta,pr}} + \frac{1}{c_{\theta,con}} \tag{4.29}$$

The stiffness of the decking may be determined from:

$$c_{\theta,de} = k \cdot \frac{EI}{a} \tag{4.30}$$

Table 4.5. Critical moments for I-beams with continuous restraint of lateral displacements

System	
Loading conditions	$-1 \leqslant \psi = \frac{M_A}{M_B} \leqslant 1,\ 0 \leqslant \mu_0 = \frac{M_B}{q \cdot l^2/8}$ M_A, M_B positive for direction of moment as shown
Critical moment	$M_{cr,B} = \frac{\pi^2 \cdot EI_w}{l^2 \cdot (h - t_f)} \cdot k_w$
Parameters	$a_T = \frac{l^2 \cdot GI_t}{\pi^2 \cdot EI_w} \quad I_w = I_z \cdot \frac{(h-t_f)^2}{4},\ k_w = a + b \cdot a_T$

Parameters a, b		$a_T \leqslant 10$		$a_T > 10$	
		a	b	a	b
	$\psi = 0$	$12 \cdot \mu_0^{-1.7}$	$-0.6 \cdot \mu_0 + 3.1$	$19 \cdot \mu_0^{-1.2}$	$1.64 \cdot \mu_0^{0.25}$
	$\psi = 0.5$	$6 \cdot \mu_0^{-1.9}$	$-1.1 \cdot \mu_0 + 3.4$	$12 \cdot \mu_0^{-1.4}$	$1.57 \cdot \mu_0^{-0.27}$
	$\psi = 1$	$3.2 \cdot \mu_0^{-1.8}$	$-1.4 \cdot \mu_0 + 3.0$	$6.4 \cdot \mu_0^{-1.5}$	$1.36 \cdot \mu_0^{-0.26}$

where:

$EI =$ is the bending stiffness of sheathing or concrete decking
$a =$ span of sheathing or decking
$k = 2$ for simply supported or two span continuous sheathing or decking
$k = 4$ for three or more span continuous sheathing or decking

The stiffness term describing profile distortion may be determined from [4.2]:

$$c_{\theta,pr} = 5770 \cdot \frac{1}{\frac{h}{t_w^3} + c \cdot \frac{b_f}{t_f^3}} \ [\text{kNm/m}] \tag{4.31}$$

where:

$c = 0.5$ for I-sections
$c = 0.5$ for C-sections, positive loading (gravity, pressure)
$c = 2.0$ for C-sections, negative loading (under-pressure)
and all other symbols as in Figure 4.12.

For values of $c_{\theta,\text{con}}$ reference is made in the literature [4.2], [4.23].

The critical moment for simply supported beams subjected to uniform loading may be determined from:

$$M_{cr} = 1.132 \cdot \frac{\pi^2 \cdot EI_z}{l^2} \cdot \sqrt{\frac{I_w}{I_z} + \frac{l^2}{\pi^2} \cdot \frac{GI_t}{EI_z} + \frac{c_\theta}{EI_z} \cdot \frac{l^4}{\pi^4}} \tag{4.32}$$

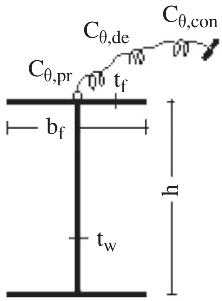


Fig. 4.12. Notation for beams with continuous torsional restraint

4.3.1.3 Numerical methods

Analytic expressions for critical lateral torsional buckling moments such as those given in the previous sections exist for members with constant cross-section under specific loading and support conditions. For single members with variable cross-section and various loading and support conditions, critical moments may be calculated on the basis of the energy method by the free software tool LTBeam developed by CTICM [4.25]. For general cases, as often appear in engineering practice, numerical methods based on FEM models may be employed and critical moments calculated by means of linear buckling analysis. However, such analysis delivers LTB modes only when structural members are represented by 7 DOF beam elements, see section 3.6.1. In addition, structural members are idealized as a line passing through the cross-section centroid so that lateral supports for one flange must be introduced as eccentric supports. In addition, 6 or 7 DOF beam elements do not include cross-section distortion effects.

As an alternative, truss models may be used for the representation of members with *I*-shaped cross-section [4.26]. The elements of the truss are a top chord, a bottom chord, connection posts and X-bracing members with following properties, Figure 4.13:

- The top chord is modelled by beam elements with T cross-section composed of the top flange of the I-section and 1/3 of the web.
- The bottom chord is modelled by beam elements with *T* cross-section composed of the bottom flange of the *I*-section and 1/3 of the web.
- The posts are beam elements of rectangular cross-section with width *s* equal to the distance between them and thickness equal to the web thickness of the *I*-section. The distance s may be selected as 5% of the member's span.
- The X-bracing members are 1 DOF truss elements that develop only axial forces. Their cross-section is rectangular with width equal to 1/3 of the web height and thickness equal to the web thickness of the *I*-section so that its area is $A_d = t_w \cdot h_w/3$.

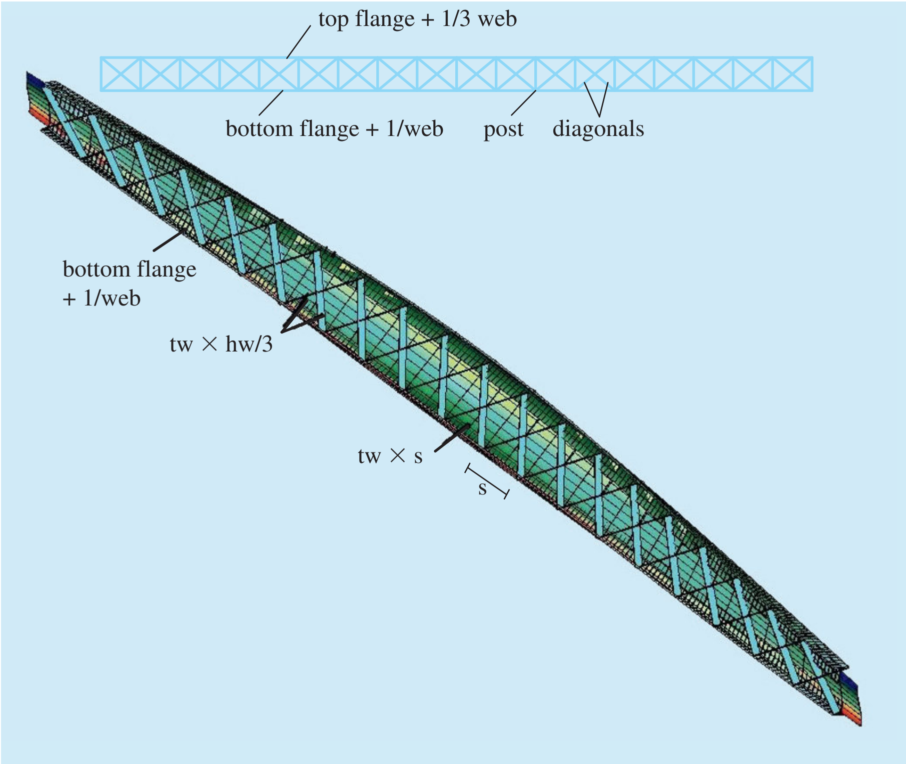


Fig. 4.13. Truss model for representation of *I*-girders

This model has been widely tested in different configurations and has been proven to be sufficiently accurate when compared with more elaborate FEM models. In addition, it is very robust in non-linear analyses including non-linear effects in respect to geometry and material as well as geometric imperfections [4.27]. Its main benefit is the possibility to consider the two flanges separately, including any additional restrain elements, and to determine directly lateral torsional buckling modes or general the non-linear response to loading, Figure 4.14. It is also able to explore with acceptable accuracy local buckling phenomena of the web in regions of high concentrated loading. The truss model is very appropriate to represent plate girders or welded *I*-sections. For rolled sections it should be modified to accommodate the rounded regions in the flange-web junction, which especially for small sections have a non-negligible contribution to their stiffness and strength. This can be done by inclusion of this area to the cross-sections of the beam elements representing the top and bottom flanges.

Numerical LBA analyses with truss or other models deliver buckling modes and corresponding critical factors α_{cr} to which applied loads should be multiplied to reach the critical state. By appropriate manipulation, critical LTB moments could then be determined. For example, the critical moment of a beam may be determined from $M_{cr} = M \cdot \alpha_{cr}$, where M is the maximum moment due to the applied loading and

α_{cr} is the smallest factor leading to a LTB mode. Numerical analyses are especially appropriate for the application of the general method as explained later.

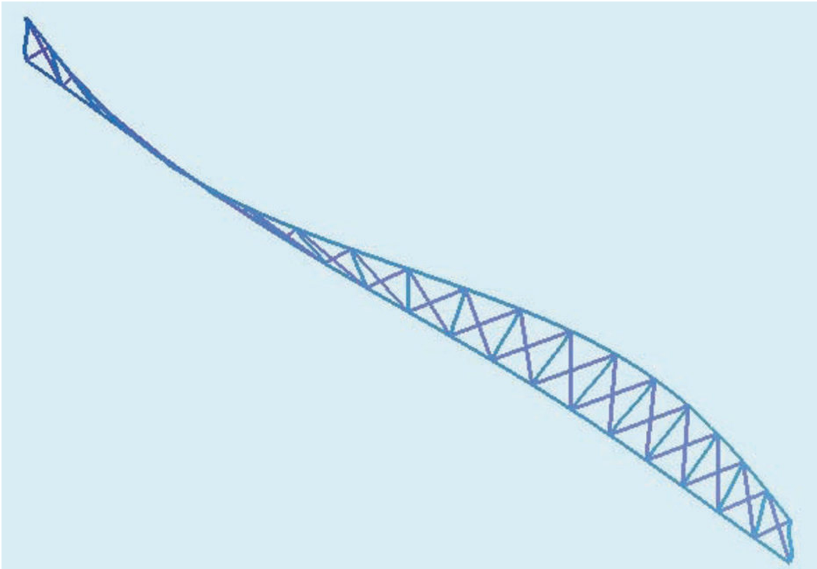


Fig. 4.14. Buckling modes of a truss model

4.3.2 LTB design moments

As the Euler buckling load does not represent the member compression capacity, the critical LTB moment does not reflect the member LTB capacity for the same reasons outlined in section 4.2.2. Here again the design LTB resistance is based on an appropriate modification of the Ayrton-Perry formula which calculates the internal forces and moment by geometrically non-linear analysis for a member subjected to strong axis bending, Figure 4.15a. If equilibrium is examined in the un-deformed state, it may be seen that due to loading only strong axis bending moments M_y develop, Figure 4.15b. However, if the system is regarded in the deformed state where a small angle of twist θ evolves due to lateral deformations of the compression flange it may be seen that the principal axes rotate so that the member is subjected to following internal moments, Figure 4.15c:

- a strong axis moment M_y, which is for small θ almost equal in both un-deformed and laterally deformed state
- a weak axis moment $M_z = M_y \cdot \theta$, applicable for small θ where sin $\theta \approx \theta$ and
- a torque T that develops due to the twist of the cross-section even if the transverse forces in the un-deformed state run through the shear center.

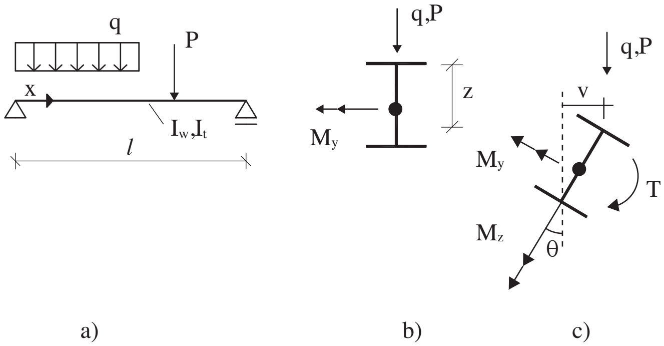


Fig. 4.15. *LTB* of beams, a) System under consideration, b) moments in the un-deformed and c) in the deformed state

By application of the Ayrton-Perry formula [4.5], a cross-section check at the most stressed section is made considering geometric non-linear effects. However, due to the absence of a compression force 2nd order effects magnify only the weak axis moments and the torque. Considering open sections that are susceptible to LTB, the torque is considered to be exclusively resisted by the warping mechanism through development of bimoments B. The cross-section check is then written as [4.28]:

$$\frac{M_y}{M_{y,R}} + \frac{M_z}{M_{z,R}} + \frac{B}{B_R} = 1 \tag{4.33}$$

where:

the nominators represent the internal moments and bimoment from 2nd order analysis and
the denominators the corresponding resistances.

More specifically it is [4.28]:

$$M_z = M_y \cdot \theta \cdot \frac{1}{1 - \frac{M_y}{M_{cr}}} \tag{4.34}$$

$$B = M_y \cdot v \cdot \frac{1}{1 - \frac{M_y}{M_{cr}}} - GI_t \cdot \theta \cdot \left(\frac{1}{1 - \frac{M_y}{M_{cr}}} - 1 \right) \tag{4.35}$$

The slenderness and the reduction factor may be similarly expressed as for compression, i.e.:

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_R}{M_{cr}}} \tag{4.36}$$

$$\chi_{LT} = \frac{M_y}{M_{y,R}} \tag{4.37}$$

The introduction of (4.34) to (4.37) in (4.33) leads to following equation for the reduction factor which is similar to eq. (4.21b):

$$\chi_{LT}^2 + \chi_{LT} \cdot \left(-1 - \frac{\eta_{LT}}{\bar{\lambda}_{LT}^2} - \frac{1}{\bar{\lambda}_{LT}^2} \right) + \frac{1}{\bar{\lambda}_{LT}^2} = 0 \tag{4.38}$$

The imperfection takes the generalized form:

$$\eta_{LT} = v \cdot \frac{M_{y,R}}{B_R} + \theta \cdot \frac{M_{y,R}}{M_{z,R}} + \theta \cdot \frac{GI_t}{M_{cr}} \frac{M_{y,R}}{B_R} \tag{4.39}$$

The solution of the above equation gives the reduction factor for LTB:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \tag{4.40}$$

where:

$$\Phi_{LT} = 0.5 \cdot (1 + \eta_{LT} + \bar{\lambda}_{LT}^2) \tag{4.41}$$

However, instead of using the complete formula (4.39) for the imperfection factor, Eurocode 3 [4.2] adopts the same expression as for flexural buckling that writes:

$$\eta_{LT} = \alpha_{LT} \cdot (\bar{\lambda}_{LT} - 0.2) \tag{4.42}$$

Introducing (4.42) in (4.41) Φ_{LT} is finally calculated from [4.2]:

$$\phi_{LT} = 0.5 \cdot [1 + \alpha_{LT} \cdot (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2] \tag{4.43}$$

Furthermore, Eurocode 3 [4.2] proposes a "specific case" for hot rolled and equivalent welded sections with following modifications to the above LTB formulas:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \cdot \bar{\lambda}_{LT}^2}} \tag{4.44}$$

where:

$\phi_{LT} = 0.5 \cdot [1 + a_{LT} \cdot (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \cdot \bar{\lambda}_{LT}^2]$
and β is a curve shape modification factor with recommended value 0.75.

In the meantime, proposals for modifying these rules for constant section or tapered beams have appeared in the literture, e.g. [4.29], [4.30].

The design procedure is summarized in Table 4.6. The Table indicates that LTB is treated in an approximative way since the geometrical imperfections are calculated from equation (4.43) that is valid for flexural buckling and not from the exact one (4.39) that corresponds to lateral torsional buckling.

Eurocode 3 proposes an alternative procedure for the specific case of rolled or equivalent welded cross-sections that is based on a modified expression for the reduction factor. This procedure is summarized in Table 4.7.

Table 4.6. Design procedure for checking bending members LTB, general case

Design format	$M_{y,Ed} \leqslant M_{b,Rd}$		(4.45)
Design resistance	$M_{b,Rd} = \chi_{LT} \cdot M_{y,pl,Rd}$ class 1 or 2 sections		(4.46a)
	$M_{b,Rd} = \chi_{LT} \cdot M_{y,el,Rd}$ class 3 sections		(4.46b)
Reduction factor	$\chi_{LT} = \dfrac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \leqslant 1$		(4.44)
	$\phi_{LT} = 0.5 \cdot (1 + a_{LT} \cdot (\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2)$		(4.43)
	$\bar{\lambda}_{LT} > 0.2$		
Relative slenderness	$\bar{\lambda}_{LT} = \sqrt{\dfrac{W_y \cdot f_y}{M_{cr}}}$		(4.47)
	$W_y = W_{y,pl}$ class 1 or 2 sections $W_y = W_{y,el}$ class 3 sections		
Critical Moment	M_{cr} from Tables 4.4, 4.5, literature or numerical methods		
Imperfection factor a_{LT}	Corresponding to European buckling curves, Table 4.2		
Assignment of cross-section to buckling curves	*Cross-section*	*Limits*	*Buckling curve*
	Rolled I sections	$h/b \leqslant 2$	a
		$h/b > 2$	b
	Welded I sections	$h/b \leqslant 2$	c
		$h/b > 2$	d
	Other		d

4.3.3 Design by the general method

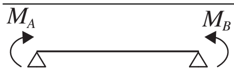
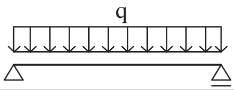
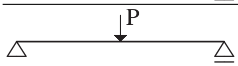
The general method is a combination of numerical/analytical procedures, applicable to both lateral and lateral torsional buckling [4.31], [4.32]. It may be used not only for checking stability of individual members in cases that are not covered by the previous analysis, like members with variable cross-sections, general loading and supporting conditions etc. but also for checking lateral stability of complete structural systems like arches, trusses etc. It requires numerical modeling of the whole 3D-structure and the performance of LBA analysis to obtain the critical load factor α_{crit}. This factor is the load amplifier of the in-plane design loads at which the fundamental buckling mode for lateral or lateral torsional buckling occurs. Subsequently the relative out-of-plane slenderness is determined from:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{\mathrm{ult},k}}{\alpha_{\mathrm{crit}}}} \tag{4.53}$$

where:

$\alpha_{\mathrm{ult},k}$ is the load amplifier of the design loads to reach the characteristic resistance of the most critical section neglecting any out-of-plane effects. If necessary, second-order bending moments should be included.
α_{crit} is the load amplifier of the in-plane design loads to reach the fundamental buckling mode for lateral or lateral torsional buckling.

Table 4.7. Design procedure for checking LTB, specific case for rolled or equivalent welded cross-sections

Design format	$M_{y,Ed} \leqslant M_{b,Rd}$			(4.45)
Design resistance	$M_{b,Rd} = \chi_{LT,\text{ mod}} \cdot M_{y,pl,Rd}$ class 1 or 2 sections			(4.48a)
	$M_{b,Rd} = \chi_{LT,\text{ mod}} \cdot M_{y,el,Rd}$ class 3 sections			(4.48b)
Reduction factor	$\chi_{LT} = \dfrac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \cdot \bar{\lambda}_{LT}^2}} \leqslant 1 \text{ and } \leqslant \dfrac{1}{\bar{\lambda}_{LT}^2}$			(4.51)
	$\phi_{LT} = 0.5 \cdot (1 + a_{LT} \cdot (\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}) + \beta \cdot \bar{\lambda}_{LT}^2)$			(4.52)
	$\bar{\lambda}_{LT0} = 0.4$ $\beta = 0.75$ (maximal value)			
Modified reduction factor (optional)	$\chi_{LT,\text{ mod}} = \dfrac{\chi_{LT}}{f} \leqslant 1$			(4.49)
	$f = 1 - 0.5 \cdot (1 + k_c) \cdot [-12.0 \cdot (\bar{\lambda}_{LT} - 0.8)^2] \leqslant 1$			(4.50)
Correction factor k_c (SS =simply supported) (C =fixed)	Loading conditions	Support conditions	k_c	
	 $-1 \leqslant \psi = \frac{M_A}{M_B} \leqslant 1$		$\dfrac{1}{1.33 - 0.33\psi}$	
		$SS - SS$	0.94	
		$C - C$	0.90	
		$SS - C$	0.91	
		$SS - SS$	0.86	
		$C - C$	0.77	
		$SS - C$	0.82	
Assignment of cross-section to buckling curves for a_{LT}	*Cross-section*	*Limits*	*Buckling curve*	
	Rolled I sections	$h/b \leqslant 2$	b	
		$h/b > 2$	c	
	Welded I sections	$h/b \leqslant 2$	c	
		$h/b > 2$	d	
	Other		d	

The reduction factor χ_{op} for lateral or lateral torsional buckling may be determined as a function of $\bar{\lambda}_{op}$ by the following condition:

$$\chi_{op} = \min(\chi, \chi_{LT}) \tag{4.54}$$

where:

χ is the reduction factor from equation (4.22) and
χ_{LT} is the reduction factor for lateral torsional buckling from equation (4.44)

The buckling verification may be written as:

$$\frac{\chi_{op} \cdot \alpha_{\text{ult},k}}{\gamma_{M1}} \geqslant 1{,}0 \tag{4.55}$$

The general method has been proven to be a very good compromise between the pure analytical and pure numerical design procedures and to the opinion of the authors it will be more widely used in the future. Its main benefits consist on it does not require the performance of full non-linear analyses nor the introduction of geometrical imperfections that need considerable experience, computational time and care. Studies showed that its application is not limited to in-plane loading but may be extended to both in- and out-of-plane loading [4.33]. α_{crit} is then the load amplifier for all design loads, while, as a simplification on the safe side, $\alpha_{\text{ult},k}$ can be based on cross-section checks that include only in-plane internal forces and moments.

4.3.4 Design by non-linear analysis

Non-linear analysis may also be employed for design of *LT* buckling cases where analytical expressions fail to provide solutions. Important for such analysis are the selection and implementation of appropriate geometric imperfections that should be considered in the members under investigation. Eurocode 3 provides two alternatives that is, theoretically, equivalent to the imperfections on which the formulae for the reduction factor χ_{LT} are based:

- Bow member imperfections towards the weak principal axis *z-z* with an amplitude $k \cdot e_{o,z}$. $e_{o,z}$ is the bow imperfection that corresponds to a buckling curve, Table 4.2. The buckling curve may be obtained from Table 4.6. The calibration factor has a recommended value $k = 0.5$.
- Imperfections that follow the first buckling mode.

The issue on appropriate geometrical imperfections for *LTB* is still under discussion, e.g. [4.34].

4.4 Members to compression and bending

4.4.1 General

Members as parts of complete structures, such as columns of portal frames, are generally subjected to compression and biaxial bending. Stability of such members may be checked individually and separately from the entire structure if they are isolated from the structure, internal forces and moments and support conditions are introduced at their ends, as well as loading p within their span, Figure 4.16.

The formulation of the equilibrium equations for such an isolated member lead to a system of coupled differential equations for which there is no analytical solution. Therefore, engineering approximate solutions in the form of interaction relationships in order to cover known simple design cases were developed and incorporated in design codes. In the following the provisions on which the Eurocode 3 [4.2] design formulae are based will be presented.

4.4.2 Magnification factors

Design expressions are usually developed considering a single simply supported member that is subjected to end forces, end moments and transverse loading, performing geometrical non-linear 2$^{\text{nd}}$ order elastic analysis and making a cross-section check at its most stressed section. Geometrical non-linear 2nd order analysis leads to magnification of bending moments only, since axial forces remain for small angles of rotation more or less unchanged, see section 2.10. The design moments may be calculated from:

$$M_{Ed}^{II} = \alpha \cdot M_{Ed}^{I} \tag{4.56}$$

where:

M_{Ed}^{I} design moments from linear, 1st order analysis
M_{Ed}^{II} design moments from geometrically non-linear 2nd order analysis
α magnification factor.

Since the latter depend on the loading conditions, their values for characteristic loading cases will be presented in the following.

4.4.2.1 Compression members with bow imperfections

This case was already presented in section 4.2.2, see Figure 4.5. The magnification factor is determined from:

$$\alpha = \alpha_1 = \frac{1}{1 - \frac{N_{Ed}}{N_{cr}}} \tag{4.57}$$

This may be rewritten as function of the "strut index" ρ, equation (4.3), as:

$$\alpha = \alpha_1 = \frac{1}{1 - \left(\frac{\rho}{\pi}\right)^2} \tag{4.58}$$

It may be seen that the magnification factor tends to infinity value when ρ tends to π. This means that the strut index has a critical, maximum, value which for this specific case is $\rho_{cr} = \pi$. Combining equations (4.57) and (4.58) this corresponds to $N_{Ed} = N_{cr}$.

4.4.2.2 Compression members with end moments

The member under consideration that is subjected to end moments M_1 and M_2 without transverse loading is shown in Figure 4.17. The moment M_2 is in absolute terms larger than the moment M_1 so that for their ratio is:

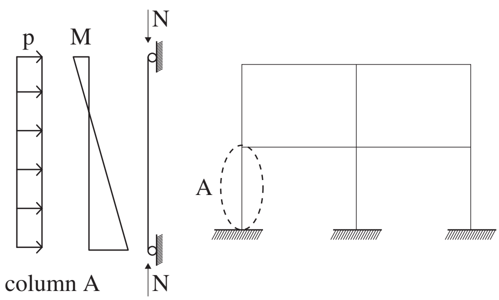


Fig. 4.16. Isolation of a member from the structural system for stability check

$$\psi = \frac{M_1}{M_2}, \quad \text{with} \quad -1 \leqslant \psi \leqslant 1 \tag{4.59}$$

The highest moment, in absolute terms, according to linear 1st order analysis is therefore equal to $\max M^I = M_2$.

In geometrically non-linear 2nd order analysis, the equilibrium condition is examined in the deformed state which may be written for the case under consideration as:

$$E\cdot I\cdot w'' + N_{Ed}\cdot w + M_1 + (M_2 - M_1)\cdot \frac{x}{l} = 0 \tag{4.60a}$$

or

$$w'' + \left(\frac{\rho}{l}\right)^2 \cdot w + \frac{M_1}{E\cdot I} + \frac{M_2 - M_1}{E\cdot I}\cdot\frac{x}{l} = 0 \tag{4.60b}$$

where ρ is the strut index, equation (4.3).

The general solution of the above differential equation may be written as:

$$w = C_1\cdot\sin\rho\frac{x}{l} + C_2\cdot\cos\rho\frac{x}{l} + C_3\cdot x + C_4 \tag{4.61}$$

The coefficients C_i, $i = 1-4$, may be determined by introduction of the boundary conditions, zero displacements – end moments, which are written as:

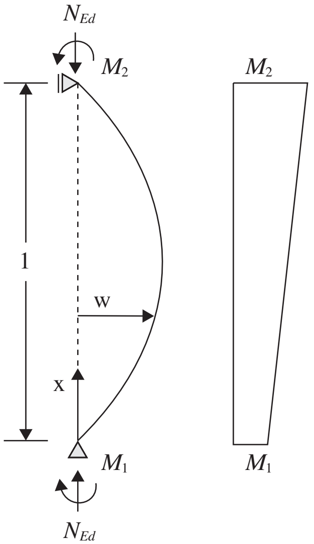


Fig. 4.17. Compression members with end moments

$$w(0) = 0 \qquad w(l) = 0 \qquad w''(0) = -\frac{M_1}{E\cdot I} \qquad w''(l) = -\frac{M_2}{E\cdot I}$$

After algebraic manipulations, the final moments at any position x along the member are determined from:

$$M(x) = -E\cdot I\cdot w''(x) = \frac{M_2 - M_1\cdot\cos\rho}{\sin\rho}\cdot\sin\rho\frac{x}{l} + M_1\cdot\cos\rho\frac{x}{l} \tag{4.62}$$

The position x_0 where the maximum moment develops is determined from the condition $\frac{dM}{dx} = 0$ as:

$$\frac{x_0}{l} = \frac{\psi\cdot\sin\rho}{\cos\rho\cdot\sqrt{1 - 2\cdot\psi\cdot\cos\rho + \psi^2}} \leqslant 1 \tag{4.63}$$

The maximum moment according to geometrically non-linear analysis is equal to:

$$\max M^{II} = \frac{M_2}{\sin\rho}\cdot\sqrt{1 - 2\cdot\psi\cdot\cos\rho + \psi^2} \tag{4.64a}$$

Obviously, when $\frac{x_0}{l} > 1$, it is $x_0 = 0$ and $\max M^{II} = M_2$.

The magnification factor, equation (4.56), is written as:

$$\alpha = \frac{\sqrt{1 - 2\cdot\psi\cdot\cos\rho + \psi^2}}{\sin\rho} \geqslant 1 \tag{4.64b}$$

For the specific case of equal end moments, it is $\psi = 1$ and the magnification factor becomes:

$$\alpha = \frac{1}{\cos\frac{\rho}{2}} \tag{4.65}$$

Figure 4.18 shows values of the magnification factors for members under bow imperfections and end moments with different values of ψ. It may be seen that the magnification factors increase exponentially with the value of ρ, as the compression force approaches the Euler buckling load. The most severe loading corresponds to application of equal end moments with same sign ($\psi = 1$), the least severe one when the moment distribution is triangular with no moment at one end ($\psi = 0$), while initial bow imperfections (α_1) result in intermediate conditions.

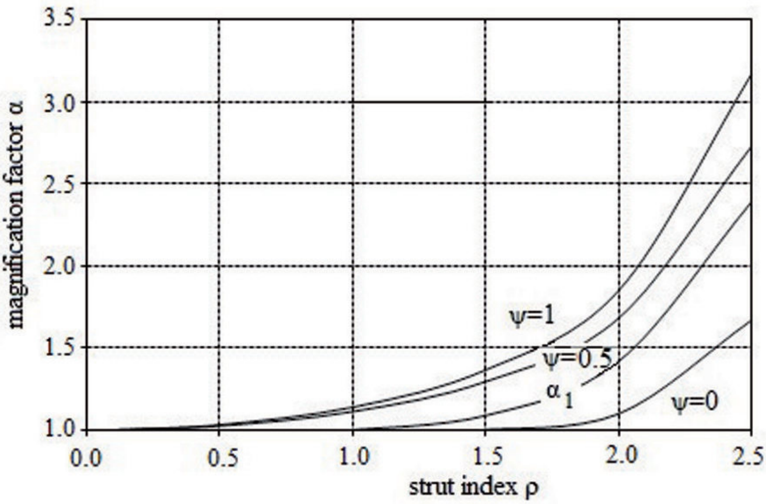


Fig. 4.18. Magnification factors for compression members with end moments and bow imperfections

4.4.2.3 Compression members under transverse loading

Moments from geometrically non-linear 2nd order analysis may be determined iteratively. In the first step moments M^I and deflections δ_1 are determined from linear 1st order analysis. These deflections, combined with the applied compression force result in additional moments $\Delta M_1 = N_{Ed} \cdot \delta_1$ and additional deflections $\Delta\delta_1$. This procedure is repeated until the deflections at two consecutive steps are almost equal, so it is $\Delta\delta_{n-1} \approx \Delta\delta_n$. Convergence is achieved provided that the compression force is smaller than the member's critical buckling load, $N_{Ed} < N_{cr}$. Table 4.8 illustrates the iterative procedure.

Table 4.8. Geometrically non-linear 2nd order analysis according to iterative procedure

Step	*Moments*	*Deflections*
0	M_1	δ_1
1	$\Delta M_1 = N_{Ed} \cdot \delta_1$	$\Delta \delta_1$
2	$\Delta M_2 = N_{Ed} \cdot \Delta \delta_1$	$\Delta \delta_2$
...	...	...
n	$\Delta M_n = N_{Ed} \cdot \Delta \delta_{n-1}$	$\Delta \delta_n$

The summation of moments from all steps provides the final moments from geometrically non-linear 2nd order analysis:

$$\begin{aligned} M^{II} &= M^I + \Delta M_1 + \Delta M_2 + \cdots + \Delta M_n = \\ &= M^I \cdot \left(1 + \frac{\Delta M_1}{M^I} + \frac{\Delta M_2}{\Delta M_1} \cdot \frac{\Delta M_1}{M^I} + \cdots + \frac{\Delta M_n}{\Delta M_{n-1}} \cdots \frac{\Delta M_1}{M^I}\right) = \\ &= M^I \cdot \left(1 + \frac{b_1}{\nu} + \frac{b_2}{\nu^2} + \cdots + \frac{b_n}{\nu^n}\right) \end{aligned} \tag{4.66}$$

where $\nu = \frac{N_{cr}}{N_{Ed}}$.

The above equation is written as:

$$M^{II} = M^I \cdot \left\{\left[(1-b_n) + \frac{b_1 - b_n}{\nu} + \frac{b_2 - b_n}{\nu^2} + \cdots\right] + b_n \cdot \left[1 + \frac{1}{\nu} + \frac{1}{\nu^2} + \cdots\right]\right\} \tag{4.67a}$$

or as approximation:

$$M^{II} = M^I \cdot \left[1 - b_n + b_n \cdot \frac{\nu}{\nu - 1}\right] = M^I \cdot \frac{1 + \delta \cdot \frac{1}{\nu}}{1 - \frac{1}{\nu}} \tag{4.67b}$$

where

$$\delta = b_n - 1 = \left(\frac{\Delta M_n}{\Delta M_{n-1}} \cdots \frac{\Delta M_1}{M^I}\right) \cdot \left(\frac{N_{cr}}{N_{Ed}}\right)^n \tag{4.68}$$

The magnification factor is accordingly equal to:

$$\alpha = \frac{1 + \delta \cdot \frac{N_{Ed}}{N_{cr}}}{1 - \frac{N_{Ed}}{N_{cr}}} \tag{4.69}$$

The introduction of the coefficient $\delta \neq 0$ as proposed by Dischinger [4.35] provides consequently a better description for the magnification factor compared to the simpler one of equation (4.57). The coefficient δ depends on the loading and boundary conditions. For a simply supported beam under concentrated mid-span loading it is $\delta = -0.18$. For uniformly distributed loading it is $\delta = 0.03$.

For more general cases b_n may be substituted by b_1 so that equation (4.67b) is written as:

$$M^{II} = M^I \cdot \left[1 - b_1 + b_1 \cdot \frac{\nu}{\nu - 1}\right] = M^I \cdot \frac{1 + (b_1 - 1) \cdot \frac{1}{\nu}}{1 - \frac{1}{\nu}}$$

The parameter b_1 is derived from equation (4.66):

$$b_1 = \frac{\Delta M_1}{M^I} \cdot \nu = \frac{N_{cr} \cdot \delta_1}{M^I} = \frac{\pi^2 \cdot E \cdot I \cdot \delta_1}{l^2 \cdot M^I}$$

Introducing the above in the equation for the 2$^{\text{nd}}$ order moment, the magnification factor is finally expressed as following:

$$\alpha = \frac{1 + \left(\frac{\pi^2 \cdot E \cdot I \cdot \delta_1}{l^2 \cdot M^I} - 1 \right) \cdot \frac{N_{Ed}}{N_{cr}}}{1 - \frac{N_{Ed}}{N_{cr}}} \tag{4.70}$$

4.4.3 Buckling of members under compression and bending

4.4.3.1 Flexural buckling in loading plane

The in-plane buckling behavior for a member with initial sinusoidal bow imperfections in the loading plane subjected to compression and uniaxial bending is examined by performance of geometrically non-linear 2$^{\text{nd}}$ order analysis, Figure 4.19. By application of the Ayrton-Perry formula [4.5], the cross-section check at the most stressed section that includes the contribution of the axial force, the moments due to imperfections and the moments due to external loading may be written as:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{\alpha_1 \cdot e_0 \cdot N_{Ed}}{M_{Rd}} + \frac{\alpha \cdot M_{Ed}}{M_{Rd}} \leqslant 1 \tag{4.71a}$$

where:

N_{Ed} is the design value of the applied axial force
M_{Ed} is the design value of the maximum bending moment along the member
N_{Rd}, M_{Rd} are the corresponding cross-section design resistances
e_0 is the maximal value of the bow imperfection with a sinusoidal distribution along the member
α_1 is the magnification factor due to imperfections, eq. (4.57)
α is the magnification factor due to external loading as applicable, section 4.4.2.

After algebraic manipulation the above expression may be written as:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{e_0 \cdot N_{Ed} + C_m \cdot M_{Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right) \cdot M_{Rd}} \leqslant 1 \tag{4.71b}$$

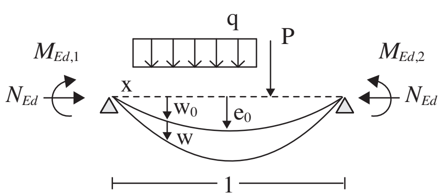


Fig. 4.19. Member with initial imperfection in the loading plane

where:

$$C_m = {}^{\alpha}/_{\alpha_1} \tag{4.72}$$

The factor C_m is expressing the ratio between the magnification factors for the loading conditions under consideration, α, and the compression member with initial imperfections, α_1. For example, for the application of end moments only and using equation (4.64b) this factor is written as:

$$C_m = \left(1 - \frac{N_{Ed}}{N_{cr,y}}\right) \cdot \frac{\sqrt{1 - 2 \cdot \psi \cdot \cos\rho + \psi^2}}{\sin\rho} \tag{4.73}$$

This expression may be substituted by a simpler linear one that is used in Eurocode 3, Method 1 and is written as:

$$C_m = 0.79 + 0.21 \cdot \psi + 0.36 \cdot (\psi - 0.33) \cdot \frac{N_{Ed}}{N_{cr,y}} \tag{4.74}$$

For pure compression it is $M_{Ed} = 0$ and the design condition becomes:

$$\frac{N_{Ed}}{\chi \cdot N_{Rd}} \leqslant 1 \tag{4.75}$$

The initial imperfection may be expressed using equations (4.71) and (4.75) as:

$$e_0 = \frac{(1-\chi) \cdot \left(1 - \frac{\chi \cdot N_{Rd}}{N_{cr}}\right)}{\chi} \cdot \frac{M_{Rd}}{N_{Rd}} \tag{4.76a}$$

or by introduction of equation (4.18b) as:

$$e_0 = \frac{(1-\chi) \cdot \left(1 - \chi \cdot \bar{\lambda}^2\right)}{\chi} \cdot \frac{M_{Rd}}{N_{Rd}} \tag{4.76b}$$

Introducing this value in equation (4.71b) the cross-section check is written as:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{(1-\chi) \cdot \left(1 - \chi \cdot \bar{\lambda}^2\right)}{\chi \cdot N_{Rd}} \cdot \frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} + \frac{C_m \cdot M_{Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right) \cdot M_{Rd}} \leqslant 1 \tag{4.77a}$$

or by use of equation (4.18b) and further algebraic manipulation as:

$$\frac{N_{Ed}}{\chi \cdot N_{Rd}} + \frac{1 - \frac{N_{Ed}}{N_{cr}}}{1 - \chi \cdot \frac{N_{Ed}}{N_{cr}}} \cdot \frac{C_m \cdot M_{Ed}}{\left(1 - \frac{N_{Ed}}{N_{cr}}\right) \cdot M_{Rd}} \leqslant 1 \tag{4.77b}$$

If the buckling check refers to the principal axis *y-y* to which the applied moments relate and by introduction of the partial safety factor for stability γ_{M1}, the design expression may be written as:

$$\frac{N_{Ed}}{\chi_y \cdot N_R / \gamma_{M1}} + k_{yy} \cdot \frac{C_{my} \cdot M_{y,Ed}}{M_{y,R} / \gamma_{M1}} \leqslant 1 \tag{4.78}$$

where:

$$k_{yy} = \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,y}}} \tag{4.79}$$

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \cdot \frac{N_{Ed}}{N_{cr,y}}} \tag{4.80}$$

Evidently, if the buckling check and applied loading refers to the weak principal axis z-z, the design expressions remain the same but the index y must be substituted by the index z.

4.4.3.2 Flexural buckling out of loading plane

For members subjected to compression and uniaxial bending, buckling may occur also out of the loading plane. This might happen if the compression force is quite large, the applied moments refer to the strong axis and are quite small and the cross-section has large difference in stiffness and strength between the two principal axes. In such cases the member may buckle along the weak axis, although the external moments are strong axis moments. To investigate this possibility, initial bow imperfections out-of the loading plane must be introduced. The design expression refers again to the most stressed cross-section and writes in analogy to equation (4.77a):

$$\frac{N_{Ed}}{N_{Rd}} + + \frac{C_m \cdot M_{z,Ed}}{(1 - \frac{N_{Ed}}{N_{cr,z}}) \cdot M_{z,Rd}} \leqslant 1 \tag{4.81}$$

or in form of equation (4.78) as:

$$\frac{N_{Ed}}{\chi_y \cdot N_R / \gamma_{M1}} + k_{yz} \cdot \frac{C_{mz} \cdot M_{z,Ed}}{M_{z,R} / \gamma_{M1}} \leqslant 1 \tag{4.82}$$

where:

$$k_{yz} = \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,z}}} \tag{4.83}$$

4.4.3.3 Lateral torsional buckling

When members are loaded along the strong principal axis and their cross-sections are susceptible to torsion, such as open sections, stability must be checked against lateral torsional buckling. Since there is no way to find analytical solutions for such cases, relations according to engineering judgment are in use. A simple way is to develop design expressions that adapt smoothly to existing expressions that apply for specific simpler cases. For example, when the compression force is missing lateral torsional buckling is checked by application of the reduction factor χ_{LT} to the bending resistance that refers to the strong axis y-y. This means that in the design check according to equation (4.78), the bending resistance $M_{y,R}$ should be substituted by the product $\chi_{LT} \cdot M_{y,R}$.

4.4.4 Member design to Eurocode 3

In absence of analytical solutions for general application, international design Codes use approximate design expressions to cover stability checks for members under general loading. Important is that design formulae should smoothly adapt to specific cases such as buckling due to compression alone or *LTB* of members subjected to pure bending. In Eurocode 3 semi-analytical design expressions were developed. These expressions include a number of parameters that were calibrated such as to deliver similar results to GMNIA analyses that were employed to buckling of members with different geometric and loading conditions. As a result, two alternative methods designated as Method 1 and Method 2 were proposed [4.36]. Method 1 [4.37] is more closely connected to analytical expressions as presented in the previous sections and therefore more transparent at first glance. However, as a result of the calibration with GMNIA analyses the parameters became so complicated that it is very difficult to comprehend their mechanical meaning apart from a result of a calibration process. On the contrary, Method 2 [4.38] did not attempt to lean too closely to analytical expressions, but fixed the parameters in a simpler way directly by calibration. Therefore, it is currently proposed to abandon completely Method 1 in the new edition of the Code. To our opinion Method 1 constitutes an example in which good intentions were destroyed by “overloading” the design expressions with so many parameters that they became eventually non-transparent. We are of the opinion that Codes should adopt simple design rules as close as possible to analytical solutions, even if deviations to numerical solutions exist which could be accommodated by higher safety factors. The benefit would be a smaller susceptibility to mistakes during application, especially when control checks are performed “by hand”.

Eurocode 3, Method 2 covers member design for elements of constant cross-section by two formulae that check member stability separately in the two principal directions and write as following [4.2]:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leqslant 1 \tag{4.84}$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leqslant 1 \tag{4.85}$$

where:

N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ design axial forces and bending moments along the principal axes of the cross-section *y-y* and *z-z*

$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ additional design moments for cross-sections of class 4 that evolve due to the shift of the cross-section centroid of the effective section from the centroid of the gross cross-section, Table 4.9.

χ_y and χ_z reduction factors due to flexural buckling, section 4.2.

χ_{LT} reduction factor due to lateral torsional buckling, section 4.3. For members sufficiently supported laterally or not susceptible to twist it is $\chi_{LT} = 1.0$

k_{yy}, k_{yz}, k_{zy}, k_{zz} interaction coefficients, Tables 4.10 and 4.11.

Table 4.9. Values for A_i, W_i and $\Delta M_{i,Ed}$

Classs of cross-section	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{\text{eff},y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{\text{eff},z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}N_{Ed}$

Table 4.10. Interaction factors k_{ij} for members not susceptible to twist

Interaction factors	*Types of cross-sections*	*Method of design*	
		Elastic design method Class 3 or 4 cross-sections	*Plastic design method Class 1 or 2 cross-sections*
k_{yy}	Torsionally restrained H-or I-sections and RHS-sections	$C_{my}\left(1+0.6\bar{\lambda}_y\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leqslant C_{my}\left(1+0.6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+(\bar{\lambda}_y-0.2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leqslant C_{my}\left(1+0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}		k_{zz}	0.6 k_{zz}
k_{zy}		0.8 k_{yy}	0.6 k_{yy}
k_{zz}	Torsionally restrained H- or I-sections	$C_{mx}\left(1+0.6\bar{\lambda}_z\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leqslant C_{mz}\left(1+0.6\frac{N_{Ed}}{\chi_z N_{RK}/\gamma_{M1}}\right)$	$C_{mz}\left(1+(2\bar{\lambda}_z-0.6)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leqslant C_{mz}\left(1+1.4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-section		$C_{mz}\left(1+(\bar{\lambda}_z-0.2)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leqslant C_{mz}\left(1+0.8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$

For I-, H- and RHS sections subjected to axial compression and uniaxial strong axis bending $M_{y,Ed}$, it may be set $k_{zy}=0$. C_m values are determined from Table 4.12.

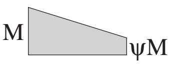
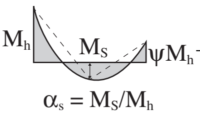
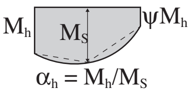
$N_{Rk}=f_y A_i$ characteristic resistance to axial forces, Table 4.9
$M_{i,Rk}=f_y W_i$ characteristic resistance to bending moments, Table 4.9

It should be noted that the $\bar{\lambda}_{LT}$-factor and the C_{mLT}-factor are related to part of the member between adjacent lateral supports. Accordingly, the ψ-factor of the moment diagram of Table 4.12 to determine C_{mLT} is different from the correspondent ψ-factor to determine C_{my} and C_{mz} in case of members with intermediate lateral restrains.

Table 4.11. Interaction coefficients k_{ij} for members susceptible to twist (unrestrained H- or I-sections)

Interaction factors	*Method of design*	
	Elastic design method *Class* 3 *or* 4 *cross-sections*	*Plastic design method* *Class* 1 *or* 2 *cross-sections*
k_{yy}	k_{yy} from Table 4.10	k_{yy} from Table 4.10
k_{yz}	k_{yz} from Table 4.10	k_{yz} from Table 4.10
k_{zy}	$\left[1-\frac{0.05\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] \geqslant \left[1-\frac{0.05}{(C_{mLT}-0.25)}\frac{N_{Ed}}{N_{Rk}/\gamma_{M1}}\right]$	$\left[-1\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] \geqslant \left[1-\frac{0.1}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ for $\bar{\lambda}_z<0.4$: $k_{zy}=0.6+\bar{\lambda}_z\leqslant 1-\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$
k_{zz}	k_{zz} from Table 4.10	k_{zz} from Table 4.10

Table 4.12. Equivalent uniform moments factors C_m in Tables 4.10 and 4.11

Moment diagram	*Range*		C_{my}, C_{mz} *and* C_{mLT} *Uniform loading*	C_{my}, C_{mz} *and* C_{mLT} *Concentrated load*
		$-1\leqslant\psi\leqslant 1$	$0.6+0.4\psi\geqslant 0.4$	$0.6+0.4\psi\geqslant 0.4$
	$0\leqslant\alpha_s\leqslant 1$	$-1\leqslant\psi\leqslant 1$	$0.2+0.8\alpha_s\geqslant 0.4$	$0.2+0.8\alpha_s\geqslant 0.4$
	$-1\leqslant\alpha_s<0$	$0\leqslant\psi\leqslant 1$	$0.1-0.8\alpha_s\geqslant 0.4$	$-0.8\alpha_s\geqslant 0.4$
		$-1\leqslant\psi<0$	$0.1(1-\psi)-0.8\alpha_s\geqslant 0.4$	$0.2(-\psi)-0.8\alpha_s\geqslant 0.4$
	$0\leqslant\alpha_h\leqslant 1$	$-1\leqslant\psi\leqslant 1$	$0.95+0.05\alpha_h$	$0.90+0.10\alpha_h$
	$-1\leqslant\alpha_h<0$	$0\leqslant\psi\leqslant 1$	$0.95-0.05\alpha_h\geqslant 0.90$	$-0.10\alpha_h$
		$-1\leqslant\psi<0$	$0.95+0.05\alpha_h(1+2\psi)$	$0.90+0.10\alpha_h(1+2\psi)$

For members with sway buckling mode it may be taken $C_{my}=0.9$ or $C_{mz}=0.9$ as appropriate

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant supported points as follows:

Moment factor	Bending axis	Point supported in direction
C_{my}	*y-y*	*z-z*
C_{mz}	*z-z*	*y-y*
C_{mLT}	*y-y*	*z-z*

4.5 Plate girders

Plate girders are fabricated sections by welding together two flange and one web plate. They are used instead of rolled beams for larger spans, higher loadings, lower fabrication costs or any particular requirements. Normally, the demand on shear capacity for beams increases proportionally to the span and on moment capacity to the square of the span. This and the necessity for higher strength to weight ratio leads to larger depths, thick flanges, thin webs and accordingly to cross-sections with class 1 to 3 flanges, class 4 webs and class 4 for the overall section. To compensate strength reductions due to plate buckling phenomena in the web, plate girders may be stiffened by transverse stiffeners, Figure 4.20, longitudinal stiffeners, or both. In the following design rules for plate girders with compact flanges and slender class 4 webs with transverse stiffeners but no longitudinal stiffeners will be presented, following the provisions of Eurocode 3, part 1-5 [4.39], as introduced in [4.40]. For girders with longitudinal stiffeners that are used mainly in bridges reference is made to the literature [4.41].

Fig. 4.20. Plate girder with transverse stiffeners

4.5.1 Resistance to bending moments

The moment resistance of plate girders with class 4 webs, and accordingly with class 4 sections, is below the elastic moment capacity of the cross-section due to plate buckling phenomena in the web. Similar to cross-section classification, plate buckling is studied separately for each cross-section wall. Figure 4.21 shows the decomposition of a plate girder cross-section into five plated elements, four external representing the flanges and one internal representing the web. Plate buckling is studied for the web plate since it is considered to be the only class 4 wall susceptible to plate buckling.

Similar to column buckling, the first step in plate buckling analysis is the determination of the elastic critical buckling stress. This is the stress at which an ideal plate without imperfections and elastic behavior becomes unstable developing out of plane deformations. Critical stresses may be determined by application of linear plate buckling theory, which considers small displacements. The critical buckling stress of plates is determined from [4.42]:

$$\sigma_{cr,p} = k_\sigma \cdot \sigma_e \tag{4.86}$$

where:

k_σ is the buckling factor and

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)(b/t)^2} = 189800 \cdot \left(\frac{t}{b}\right)^2 \quad \text{[MPa] is the reference stress} \tag{4.87}$$

In (4.87) b is the width and t is the thickness of the plate.

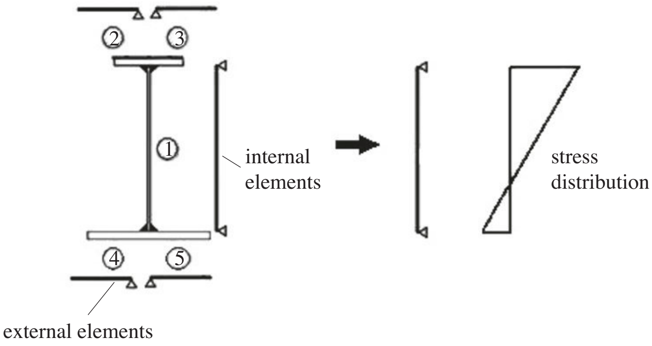
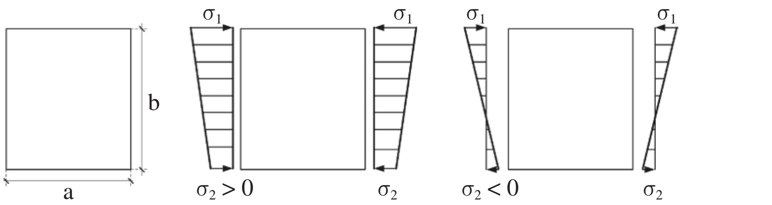


Fig. 4.21. Decomposition of cross-section walls for classification and plate buckling analysis

Buckling factors depend on the support and loading conditions and are illustrated in Table 4.13 for internal panels.

Table 4.13. Buckling factors k_σ for internal panels [4.39]

Distribution of direct stresses (compression positive)



$\psi = \dfrac{\sigma_2}{\sigma_1}$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4.0	$8.2/(1.05 + \psi)$	7.81	$7.81 - 6.29\psi + 9.78\psi^2$	23.9	$5.98 \cdot (1 - \psi)^2$

Like in struts, the ultimate strength of plates differs from the critical buckling strength. The difference between critical stresses and ultimate strength is due to the fact that against the assumptions made for defining critical stresses, real plates have geometric imperfections, structural imperfections (initial welding stresses) due to the fabrication processes and steel material is not indefinitely elastic. In addition, plates possess considerable post-buckling strength. The ultimate strength is determined by application of a reduction factor to the material yield strength. In analogy to struts, a relative slenderness is defined for plates from:

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr,p}}} \tag{4.88}$$

Buckling curves provide reduction factors as a function of the relative slenderness which are defined for internal panels from following expression:

$$\rho = \frac{\bar{\lambda}_p - 0.055\,(3+\psi)}{\bar{\lambda}_p^2} \leqslant 1 \tag{4.89}$$

where ψ is the stress ratio defined in Table 4.13.

The moment capacity of plate girders could be conservatively determined by checking the plate buckling strength of its web. Instead, the effective width method is used for design that allows shedding of stresses between cross-section walls, which results in an enhanced resistance of the complete cross-section. The effective width method starts from the observations that the stress distribution across a panel is non-uniform in the post-buckling range due to shedding of stresses from the middle part that have buckled to the stiffer edges, Figure 4.22. In the effective width model [4.42], [4.43] the non-uniform stress distribution over the entire width b is substituted by a uniform stress distribution over a reduced effective width b_{eff}, a procedure similar to the effective width model that is introduced due to shear lag effects.

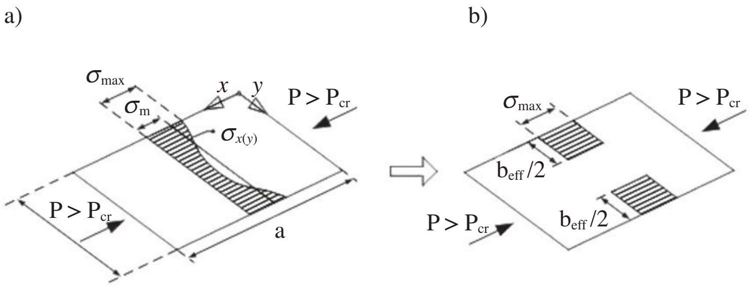


Fig. 4.22. Stress distribution: a) at post-buckling state and b) according to effective width

The effective width is determined from the condition that the acting axial force is equal in both cases and is given by:

$$b_{\text{eff}} = \rho \cdot b \leqslant b \tag{4.90}$$

where:

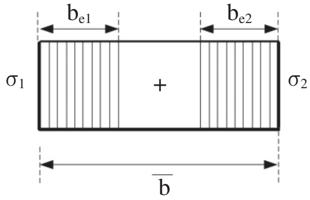
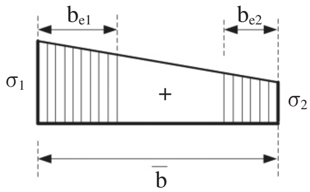
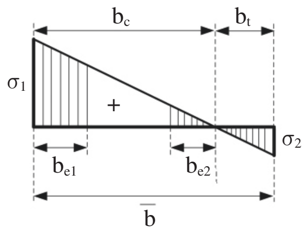
ρ is the reduction factor due to plate buckling, eq. (4.89) and
b is the entire width.

It is remarked that if the maximum design stress $\sigma_{com,Ed}$ is smaller than the yield strength f_y, the effective width may be enhanced due to the fact that the relative slenderness may be reduced according to:

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_y/\gamma_{M0}}} \tag{4.91}$$

Table 4.14 presents the effective width of internal panels which is obviously introduced only for class 4 walls where $\rho < 1$.

Table 4.14. Effective widths for internal elements [4.39]

Stress distribution (compression positive)	*Effective width b_{eff}*
	$\psi = 1$: $b_{\text{eff}} = \rho \cdot \bar{b}$ $b_{e1} = 0.5 \cdot b_{\text{eff}}$ $b_{e2} = 0.5 \cdot b_{\text{eff}}$
	$1 > \psi \geqslant 0$: $b_{\text{eff}} = \rho \cdot \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} \cdot b_{\text{eff}}$ $b_{e2} = b_{\text{eff}} - b_{e1}$
	$\psi < 0$: $b_{\text{eff}} = \rho \cdot b_c = \dfrac{\rho \cdot \bar{b}}{1-\psi}$ $b_{e1} = 0.4 \cdot b_{\text{eff}}$ $b_{e2} = 0.6 \cdot b_{\text{eff}}$

Note
$\psi = \sigma_2 / \sigma_1$

Having introduced effective widths for the class 4 web panel, a new effective cross-section evolves, Figure 4.23. The bending resistance is based on the properties of the effective cross-section and is determined from:

$$M_{Rd} = W_{\text{eff}} \cdot \frac{f_y}{\gamma_{M0}} \tag{4.92}$$

Cross-section verification is then performed according to following relation:

$$\eta_1 = \frac{M_{Ed}}{W_{\text{eff}}} \frac{\gamma_{M0}}{f_y} \leqslant 1 \tag{4.93}$$

For web panels with variable stresses along their length the verification of Eq. (4.93) should be done at a distance $s = \min\{0.4 \cdot a \text{ or } 0.5 \cdot b\}$ of the most stressed panel end, where a is the length of the panel (distance between transverse stiffeners) and b the width of the panel (height of the web). In addition, a check at the end of the panel using gross section properties should be done.

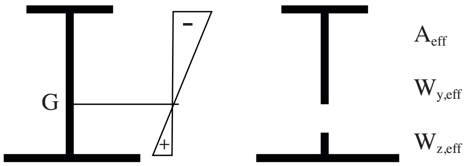


Fig. 4.23. Cross and effective cross-section

4.5.2 Resistance to shear

Slender web panels of plate girders are prone to shear buckling due to shear forces. Design to shear is analogous to moments. The critical buckling shear stress is given by:

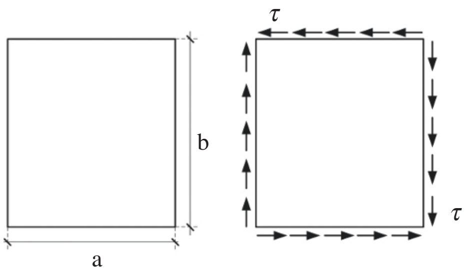
$$\tau_{cr} = k_\tau \cdot \sigma_e \tag{4.94}$$

where:

k_τ is the shear buckling factor, Table 4.15 and
σ_e the reference stress, eq. (4.87)

Table 4.15. Shear buckling factors k_τ [4.39]

Shear stresses



Aspect ratio	$\alpha = \frac{a}{b} \geqslant 1$	$\alpha = \frac{a}{b} < 1$
Buckling Factor k_τ	$k_\tau = 5.34 + \frac{4.0}{\alpha^2}$	$k_\tau = 4.0 + \frac{5.34}{\alpha^2}$

The relative slenderness is determined from:

$$\bar{\lambda}_w = \sqrt{\frac{f_y/\sqrt{3}}{\tau_{cr}}} = 0.76 \cdot \sqrt{\frac{f_y}{\tau_{cr}}} \tag{4.95}$$

or taking $k_\tau = 5.34$ which is valid for high aspect ratios α (no or a few transverse stiffeners):

$$\bar{\lambda}_w = \frac{b}{86.4 \cdot t \cdot \varepsilon} \tag{4.96}$$

The reduction factors for shear buckling are given in Table 4.16 which implies that a reduction in shear strength due shear buckling shall be accounted for slenderness $\bar{\lambda}_w > 0.83/\eta$, or considering equation (4.96) when (in parenthesis values for steel grades equal or higher than S 460):

$$\frac{h_w}{t} > 60 \cdot \varepsilon \qquad \left(\frac{h_w}{t} > 72 \cdot \varepsilon\right) \tag{4.97}$$

The shear buckling resistance considering only the contribution of the web and neglecting it of the flanges is determined from:

$$V_{b,Rd} = \frac{\chi_w\, f_{yw}\, h_w\, t}{\sqrt{3}\, \gamma_{M1}} \tag{4.98}$$

where:

χ_w is the reduction factor for shear as determined from Table 4.16.
f_{yw} is the yield strength of the web
and the other notation as shown in Figure 4.24.

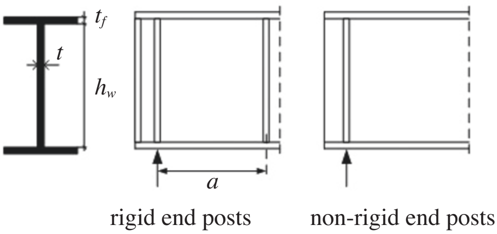


Fig. 4.24. Notations and types of support conditions

Table 4.16. Reduction factors χ_w for shear buckling resistance [4.39]

Range	*Rigid end posts, Fig.* 4.24	*Non-rigid end posts, Fig.* 4.24
$0.83/\eta > \bar{\lambda}_w$	η	η
$0.83/\eta \leqslant \bar{\lambda}_w < 1.08$	$0.83/\bar{\lambda}_w \leqslant 1$	$0.83/\bar{\lambda}_w \leqslant 1$
$\bar{\lambda}_w \geqslant 1.08$	$1.37/\left(0.7+\bar{\lambda}_w\right)$	$0.83/\bar{\lambda}_w \leqslant 1$

Notes:
EN1993-1-5 recommends $\eta = 1.2$. For steel grades higher than S460 $\eta = 1.0$.

The verification for shear is performed according to following relation:

$$\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leqslant 1 \tag{4.99}$$

It is noted that the flanges may contribute to the shear resistance besides the web. For more information, reference is made to the literature.

4.5.3 Interaction of bending and shear

Plate girders should be checked separately for bending and shear as described in the previous sections. In addition, the interaction between bending moments and shear should be checked in cases where $\eta_3 > 0.5$ by the following relation:

$$\overline{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)(2 \cdot \eta_3 - 1)^2 \leqslant 1.0 \tag{4.100}$$

where:

$$\overline{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$M_{pl,Rd}$ is the plastic moment of the cross-section, although it is class 4.
$M_{f,Rd}$ is the design bending resistance of the cross-section consisting of the flanges only.

4.6 Built-up compression members

4.6.1 Critical buckling load

For high loads built-up members may be used, where the parallel chords composed by two rolled or welded parallel sections, and designated as chords are connected with lacings or battening, Figure 4.25. Such members may be modeled either as framed elements, where each structural element is introduced separately or as a single Timoshenko beam [4.44] with equivalent properties in which the influence of shear deformations is significant as outlined in the following.

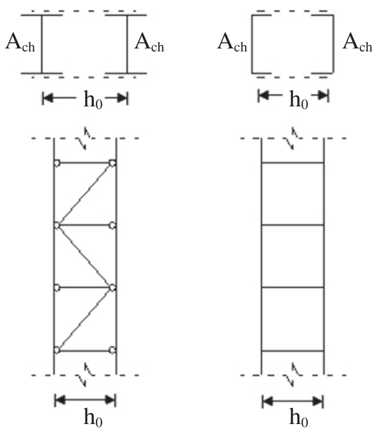


Fig. 4.25. Examples of laced or battened built-up members

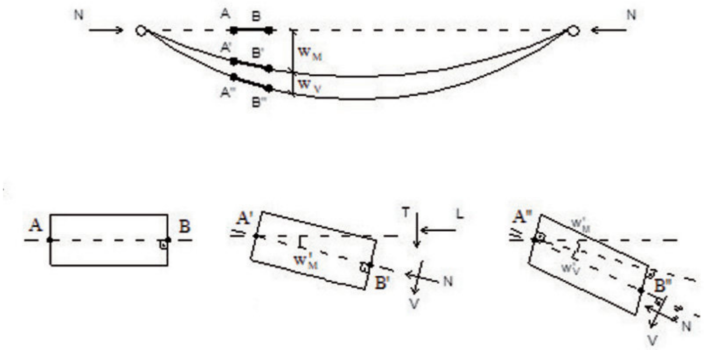


Fig. 4.26. Buckling of member with shear deformations

Equilibrium conditions of an axially compression member in the deformed state may be written as, Figure 4.26:

$$EI \cdot w'' + N \cdot w = 0 \rightarrow w''_M = -\frac{N}{EI} \cdot w \tag{4.101a}$$

where the index M indicates that only deformations due to bending were considered. For the Timoshenko beam with shear stiffness S_v, shear deformations are also taken into account considering following relations:

$$w'_v = \frac{V}{S_v} = \frac{N \cdot w'}{S_v} \rightarrow w''_v = \frac{N}{S_v} \cdot w'' \tag{4.101b}$$

Combining the above equations, the differential equation of the Timoshenko beam may be derived as:

$$w'' = w''_M + w''_v = -\frac{N}{EI} \cdot w + \frac{N}{S_v} \cdot w'' \rightarrow w'' + \frac{N}{EI \cdot \left(1 - \frac{N}{S_v}\right)} \cdot w = 0 \tag{4.101c}$$

The critical load is then determined from:

$$\frac{N_{cr}}{EI \cdot \left(1 - \frac{N_{cr}}{S_v}\right)} = \frac{\pi^2}{l^2} \tag{4.102a}$$

The critical Euler load for the same column neglecting shear deformations is determined from following relation, where the effective second moment of area I_{eff} is determined from (4.105) and (4.106):

$$N_E = \frac{\pi^2 \cdot EI_{\text{eff}}}{l^2} \tag{4.102b}$$

The critical buckling load of the Timoshenko beam is finally written as:

$$N_{cr} = \frac{N_E}{1 + \frac{N_E}{S_v}} = \frac{1}{\frac{1}{N_E} + \frac{1}{S_v}} \tag{4.103}$$

It may be observed that for a member with infinite bending rigidity, $EI = \infty$, the critical buckling load is equal to its shear stiffness according to equation (4.104), whereas its buckling form is a straight line:

$$N_{cr} = S_v \tag{4.104}$$

The shear stiffness S_v of laced and battened built-up members is given in Table 2.5. The stiffness in that table refers to one plane of lacings or battens while for spatial structures it must be multiplied by n, where n is the number of planes.

The effective second moment of area I_{eff} for built-up members may be determined from:

- Laced built-up members:

$$I_{\text{eff}} = 0.5 \cdot h_0^2 \cdot A_{ch} \tag{4.105}$$

- Battened built-up members:

$$I_{\text{eff}} = 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot \mu \cdot I_{ch} \tag{4.106}$$

where, Figure 4.25:

h_0 is the axial distance between chords
A_{ch} is the cross-section area of one chord
I_{ch} is the in plane second moment of area of one chord
μ is an efficiency factor from Table 4.17.

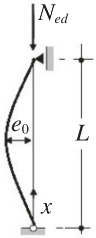
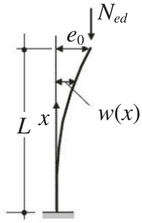
Table 4.17. Efficiency factor μ for battened built-up columns [4.2]

Slenderness λ	*Efficiency factor* μ
$\lambda \leqslant 75$	$\mu = 1.0$
$75 < \lambda < 150$	$\mu = 2 - \frac{\lambda}{75}$
$150 \leqslant \lambda$	$\mu = 0$
$\lambda = \frac{L}{i_0} \left(= \frac{2L}{i_0} \text{ for cantilevers} \right)$	$i_0 = \sqrt{\frac{I_1}{2 \cdot A_{ch}}}$ $I_1 = 0.5 \cdot h_0^2 \cdot A_{ch} + 2 \cdot I_{ch}$

4.6.2 Internal forces and moments and design

The internal forces and moments of a single compression member with equivalent properties in respect to bending and shear, EI and S_v, may be determined by geometrical non-linear 2^{nd} order analysis with geometrical bow imperfections. Table 4.18 presents results for columns with pinned supports and for cantilevers.

Table 4.18. Maximal moments and shear for the equivalent single member due to compression force

System		
max imperfection $e_0 =$	$L/500$	$2 \cdot L/500$
Imperfection $w_0(x) =$	$e_0 \cdot \sin \frac{\pi x}{L}$	$e_0 \cdot \cos \frac{\pi x}{2 \cdot L}$
Moment $M_{Ed}(x) =$	$\frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} \cdot e_0 \cdot \sin \frac{\pi x}{L}$	$\frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} \cdot e_0 \cdot \cos \frac{\pi x}{2 \cdot L}$
Shear force $V_{Ed}(x) = M'(x) =$	$\frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} \cdot e_0 \cdot \frac{\pi}{L} \cdot \cos \frac{\pi x}{L}$	$\frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} \cdot e_0 \cdot \frac{\pi}{2 \cdot L} \cdot \sin \frac{\pi x}{2 \cdot L}$
$\max M_{Ed} =$	at $x = L/2$	at $x = 0$
$\max V_{Ed} =$	at $x = 0$ and $x = L$	at $x = 0$
$N_{cr} =$	$\left(\frac{1}{N_E} + \frac{1}{S_v}\right)^{-1}$	
Note The above moments and shear forces are due to the compression force. Their final values are provided by adding up the corresponding moments and shear forces due to transverse loading.		

The moments and shear forces from Table 4.18 refer to the equivalent beam and must be distributed for design to the individual elements of the built-up member such as the chords, diagonals and battens. More specifically the design of built-up members is as follows:

- Laced built-up members

Chords and diagonals of laced built-up members are designed for buckling due to axial compression. More specifically, the design forces and corresponding resistances are determined as follows.

- Design for chords

$$N_{ch,Ed} \leqslant N_{b,Rd} \tag{4.107}$$

where:

$N_{ch,Ed}$ is the design compression force of the chord determined from:

$$N_{ch,Ed} = \frac{N_{Ed}}{2} + \frac{\max M_{Ed} \cdot h_0 \cdot A_{ch}}{2 \cdot I_{\text{eff}}} \tag{4.108}$$

$N_{b,Rd}$ is the design buckling resistance of the chord, with buckling length of the chord from Figure 4.27.
$\max M_{Ed}$ is the maximum moment of the equivalent member due to compression force and possible transverse loading, Table 4.18 and
I_{eff} the effective second moment of area of the chord, equation (4.105) and all other symbols as in Figure 4.25.

- Design for diagonals

$$N_{d,Ed} \leqslant N_{b,Rd} \tag{4.109}$$

where:

$N_{d,Ed}$ is the design compression force of the diagonal determined from:

$$N_{d,Ed} = \frac{\max V_{Ed}}{n \cdot \cos\alpha} \tag{4.110}$$

$N_{b,Rd}$ is the design buckling resistance of the diagonal, with buckling length equal to the system length
$n = 2$, number of planes
α is the angle of the diagonal to the vertical, Figure 4.27.

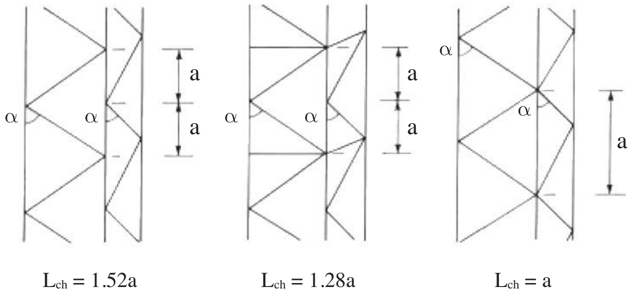


Fig. 4.27. Buckling length of the chords of laced built-up members

- Battened built-up members

Battened built-up members are designed as Vierendeel beams considering fictitious hinges in the middle of chords and battens, Figure 4.28

- Design for chords

Chords of battened built-up member are designed for flexural buckling due to compression and bending.

The compression force is determined from equation (4.108).

The bending moment is due to the shear force in the equivalent member and is determined from:

$$M_{ch,Ed} = \frac{V_{Ed} \cdot a}{n \cdot 4} \tag{4.111}$$

where:

V_{Ed} is the shear force of the equivalent member from Table 4.18
a is the axial distance between battens.

The buckling length is equal to a.

- Design for battens

Battens are subjected to cross-section check due to bending moments and shear forces.

The bending moments are two times the moments of the chords and are equal to:

$$M_{bat,Ed} = \frac{V_{Ed} \cdot a}{n \cdot 2} \tag{4.112}$$

The shear force is the vertical force in the hinge and is determined from:

$$V_{bat,Ed} = \frac{M_{bat,Ed}}{h_0} \tag{4.113}$$

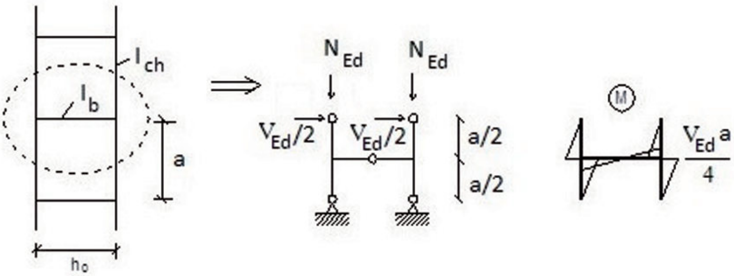


Fig. 4.28. Forces at elements of battened built-up members

4.7 Composite beams

Steel beams connected with mechanical fasteners to the top concrete slab form composite steel-reinforced concrete beams. The connection between the concrete slab and the underlying top flange of the steel beam restrains the slip between the contact surfaces and is provided by mechanical connectors such as shear studs. Composite beams are the usual beam type for buildings with concrete slab floors. Steel bridges with concrete decks also belong to composite construction. Similar to steel beams, composite beams shall be verified for cross-section and member capacity and additionally for longitudinal shear to ensure the composite action. In the following design of composite beams for buildings in accordance with the provisions of Eurocode 4 [4.45] will be briefly presented, focusing on differences to pure steel beams. The text covers the most usual cases, for more details reference is made to the literature [4.46], [4.47].

4.7.1 Resistance to bending moments

Composite beams behave most advantageously when subjected to hogging bending where the concrete is in compression. The beam section is composed of the concrete flange and the steel beam. For the bending capacity the geometric width b of the concrete flange is possibly reduced to the effective width, b_{eff}, due to the influence of shear lag, Figure 4.29. The effective width may be taken as $L_e/4$, but $b_{\text{eff}} \leqslant b$, where L_e is the distance between zero bending moments in the beam. For simply supported beams $L_e = L$, where L is the span. For spans of continuous beams $L_e = 0.85 \cdot L$ for external spans and $L_e = 0.70 \cdot L$ for internal spans, while over the supports L_e is $^1/_4$ of the sum of the adjacent spans.

The bending moment resistance of the composite beam is determined by splitting the moment in a pair of forces, where the concrete flange is in compression and the steel beam in tension.

The full compression capacity of the concrete flange is equal to:

$$F_{c,Rd} = b_{\text{eff}} \cdot d \cdot 0.85 \cdot f_{cd} \tag{4.114}$$

where:

b_{eff} is the effective width of the concrete flange
d is the thickness of the concrete slab
$f_{cd} = \frac{f_{ck}}{\gamma_c}$ is the design compression strength of concrete
f_{ck} is the characteristic compression strength of concrete, Table 4.19.
$\gamma_c = 1.5$ is the partial safety factor for concrete.

The full tension capacity of the steel beam is equal to:

$$F_{t,Rd} = A_a \cdot f_{yd} \tag{4.115}$$

where:

A_a is the cross-section area of the steel beam
$f_{yd} = \frac{f_{yk}}{\gamma_{M0}}$ is the design yield strength of steel
$\gamma_{M0} = 1.0$ is the partial safety factor for yielding of steel.

The compression and tension capacities are mutually compared in order to determine the plastic neutral axis. Usually in buildings it is $F_{c,Rd} \geqslant F_{t,Rd}$, on which case the plastic neutral axis is within the concrete flange. Its position, expressing the depth of

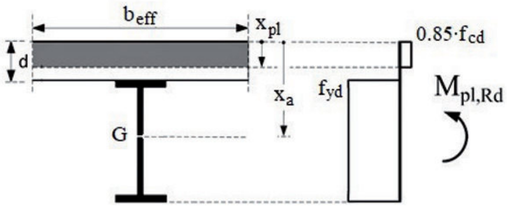


Fig. 4.29. Composite beams to hogging bending

concrete under compression, Figure 4.29, is determined by equalizing the compression and tension forces:

$$x_{pl} = \frac{A_a \cdot f_{yd}}{b_{\text{eff}} \cdot 0.85 \cdot f_{cd}} \quad (4.116)$$

The plastic resistance to hogging moments is then determined from:

$$M_{pl,Rd} = A_a \cdot f_{yd} \cdot (x_a - 0.5 \cdot x_{pl}) \quad (4.117)$$

where:

x_a is the distance of the steel beam's centroid from the top flange

Figure 4.29 shows that the entire steel section is in tension. Accordingly, no local buckling occurs, the cross-section is class 1 and the moment resistance was correctly determined as the plastic resistance of the cross-section.

For other cases, not so usual in buildings, in which the neutral axis is within the steel section, as well as for the determination of the bending resistance to hogging moments reference is made to the literature.

Table 4.19. Properties of concrete

Grade	*C*20/25	*C*25/30	*C*30/37	*C*35/45	*C*40/40	*C*45/55	*C*50/60	*C*55/67	*C*60/75
f_{ck} [MPa]	20	25	30	35	40	45	50	55	60
E_{cm} [GPa]	30	31	33	34	35	36	37	38	39

The plastic moment resistance defined by equation (4.117) is achieved for full shear connection as described in sections 4.7.3. Evidently, the bending capacity is determined by the capacity of the steel girder alone, $M_{pl,a,Rd}$, if there is no shear connection between the steel girder and the concrete slab. In practice the shear connection may be between the two extremes – no connection, full connection – a condition that is called partial shear connection. The degree of partial shear connection is expressed by the parameter η defined by equation (4.123). Partial shear connection influences the bending capacity that is between $M_{pl,a,Rd}$ for no connection to $M_{pl,Rd}$ for full shear connection. A simple means to calculate the bending resistance for partial shear connection is to interpolate linearly between the two extreme values as illustrated in Figure 4.30.

4.7.2 Resistance to vertical shear

Shear forces are supposed to be resisted by the steel section alone so that reference is made to sections 3.5 and 4.5.2 of this book concerning shear and shear buckling resistance.

4.7.3 Shear connection

Shear connection ensures composite action by restraining slip between the contact surfaces of the girder top flange and the bottom part of the concrete slab. Considering

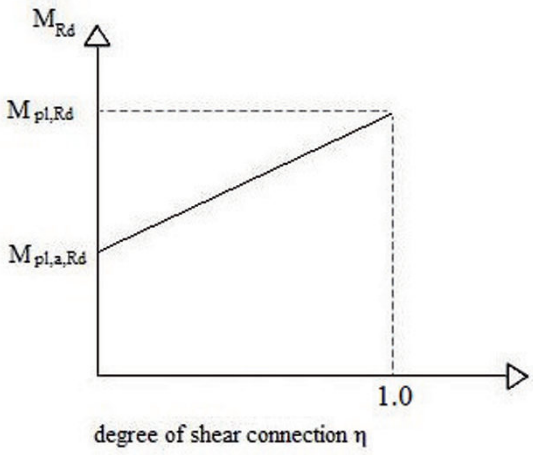


Fig. 4.30. Moment resistance for partial shear connection

the free body diagram of the concrete slab between cross-sections of zero moment "0" and full plastic moment "m" it may be seen that the compression force in the slab F_c is resisted by the longitudinal shear F_l that develops at the interface between the steel beam and the slab, Figure 4.31. This shear force should accordingly be resisted by shear connectors that are welded to the top flange of the beam and are encased in the concrete slab.

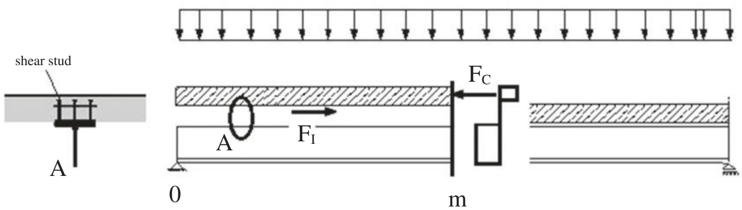


Fig. 4.31. Free body diagram of the concrete slab and longitudinal shear force

The most usual shear connectors are headed studs. Their failure appears in two distinct modes and the relevant resistances for solid slabs are determined as follows:

- Failure mode 1: Shear at shank toe

$$P_{Rd,1} = \frac{0.8 \cdot f_u \cdot \left(\pi \cdot d^2/4\right)}{\gamma_v} \tag{4.118}$$

- Failure mode 2: Crushing of concrete around the shank

$$P_{Rd,2} = \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v} \tag{4.119}$$

where:

d is the diameter of the shank of the stud, but 16 mm $\leqslant d \leqslant$ 25 mm and $d \leqslant 2.5 \cdot t_{ao}$, ($t_{ao}$ is the thickness of the top steel flange)
f_u is the specified nominal strength of the stud material but $\leqslant$ 500 MPa
f_{ck} is the cylinder strength of concrete, Table 4.19
E_{cm} is the mean value of the modulus of elasticity of concrete, Table 4.19
$\gamma_v = 1.25$ is the partial safety factor of resistance

$$\alpha = 0.2 \cdot \left(\frac{h_{sc}}{d} + 1\right) \; for \; 3 \leqslant \frac{h_{sc}}{d} \leqslant 4$$

$$\alpha = 1 \text{ for } \frac{h_{sc}}{d} > 4$$

h_{sc} is the height of the stud.

The final design resistance is provided as the minimum value of the two failure modes from:

$$P_{Rd} = \min\left\{P_{Rd,1}, P_{Rd,2}\right\} \tag{4.120}$$

For composite slabs with metal decking and top concrete design resistances are determined by application of appropriate reduction factors to the above resistance values for which reference is made to the Code [4.45].

For full shear connection the number of required shear studs within the critical length defined as the distance between zero and maximal moments, sections 0 and m in Figure 4.31, is determined by dividing the longitudinal shear by the capacity of one stud and is given by:

$$n_f = \frac{F_{l,Ed,pl}}{P_{Rd}} \tag{4.121}$$

where:

$$F_{l,Ed,pl} = \min(F_{c,Rd}, F_{t,Rd}) \tag{4.122}$$

$F_{c,Rd}$ is the full compression resistance of the concrete flange, equation (4.114)
$F_{t,Rd}$ is the plastic tension resistance of the steel beam, equation (4.115).

If the actual number of shear studs is less than n_f, the shear connection is partial. The degree of shear connection is defined by the parameter η calculated from:

$$0 \leqslant \eta = \frac{n}{n_f} \leqslant 1 \tag{4.123}$$

where:

n is the actual number of shear studs within the critical length and
n_f is the number of shear studs for full shear connection, equation (4.121)

However, the degree of shear connection may not be too low for a beam to be considered as composite. The minimum degree of shear connection in buildings must be limited according to equations (4.124) and (4.125), which are valid when ductile shear connectors are used. Headed studs fulfilling the diameter limitations indicated below equation (4.119) are considered as ductile shear connectors.

- Steel beams with equal flanges:

$$L_e \leqslant 25 \text{ m} \qquad \eta \geqslant \max\left\{\left[1-\left(\frac{355}{f_y}\right)\cdot(0.75-0.03\cdot L_e)\right];[0.4]\right\} \tag{4.124}$$

$$L_e > 25 \text{ m} \qquad \eta \geqslant 1 \tag{4.125}$$

where:

f_y is the yield strength of the steel beam in [MPa].
L_e is the distance between zero moments in sagging bending of the beam
$= L$ (L is the span) for simply supported beams,
$= 0.85 \cdot L$ for external spans of continuous beams
$= 0.70 \cdot L$ for internal spans of continuous beams

- Steel beams with $A_{fo} = 3 \cdot A_{fu}$ (A_{fo},A_{fu} area of the top correspondingly bottom steel beam flange):

In equation (4.124) the factor 0.03 becomes 0.015.

- Steel beams with $A_{fu} \leqslant A_{fo} \leqslant 3 \cdot A_{fu}$:

Linear interpolation of the factor between 0.03 and 0.015

- Steel beams of frames in seismic areas that are part of the system providing seismic resistance:

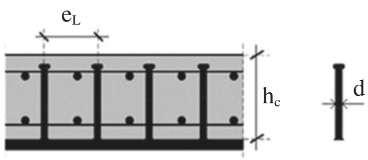
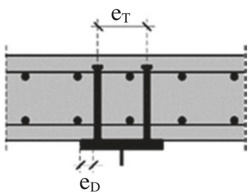
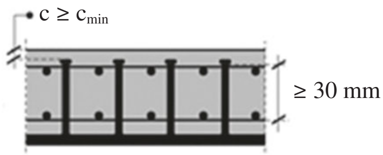
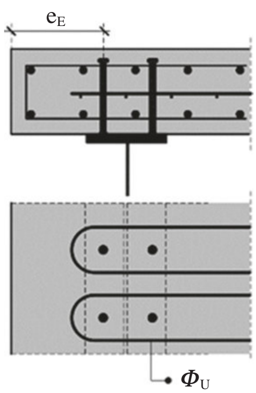
$$\eta \geqslant 0.8$$

The distribution of shear connectors along the critical length may be uniform, meaning equal number in cross-sections and equal spacing in longitudinal direction, provided that:

- The shear connectors are ductile, such as headed studs with limitations in diameter specified above,
- all critical sections in the span considered are class 1 or 2, a condition that is always satisfied for simply supported beams
- η satisfies the limits given above
- $M_{pl,Rd} \leqslant 2.5 \cdot M_{pl,a,Rd}$, where the left part is the plastic resistance of the composite beam and the right part the plastic resistance of the steel beam.

Alternatively, or when the above conditions are not met the shear connectors must be designed by elastic analysis and be spaced such as to cover the design longitudinal shear forces. Figure 4.32 shows an example of elastic design, indicating that the diagram of the design longitudinal shear $v_{L,Ed}$ to be covered is affine to the diagram of vertical shear forces.

Table 4.20. Detailing rules for shear studs, solid slabs [4.45]

Condition	*Limitation*
Spacing in longitudinal direction (e_L)	 $5 \cdot d \leqslant e_L \leqslant \min\{4 \cdot h_c, 800\text{ mm}\}$
Spacing in transverse direction (e_T) and clear distance between edge of stud and edge of flange (e_D)	 $e_D \geqslant 25$ mm For solid slabs $e_T \geqslant 2.5 \cdot d$ else $e_T \geqslant 4.0 \cdot d$
Studs on compression flanges that would be class 3 or 4 but are classified due to the shear connection as 1 or 2	$e_L \geqslant 22 \cdot \varepsilon \cdot t_{ao}$ for solid slabs and $e_D \geqslant 9 \cdot \varepsilon \cdot t_{ao} \quad \varepsilon = \sqrt{\frac{f_y}{235}}$ t_{ao} = thickness of top steel flange
– Distance between down side of head and transverse slab reinforcement – Concrete cover for shear connectors (c) If cover is not required, a zero cover is allowed $(c = 0)$. 	$\geqslant 30$ mm for flat slabs $c \geqslant \max(20\,\text{mm}, \text{acc. to EN 1992-1-1})$
Prevention of longitudinal splitting of concrete in edge girders. If the distance of the edge of the concrete flange to the centerline of the nearest row of shear connectors (e_E) is less than 300 mm then additional U-bars passing around the shear connectors of the edge girders should be provided.	 $e_E \geqslant 6 \cdot d$ $\Phi_U \geqslant 0.5 \cdot d$

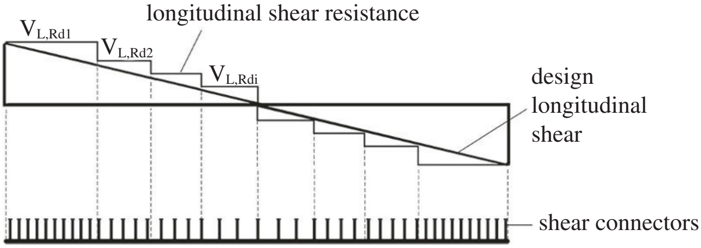


Fig. 4.32. Elastic design of shear connectors by cover of the longitudinal shear

There exist certain detailing rules in respect to the spacing of shear connectors in longitudinal and transverse direction, the distance between transverse reinforcement and the lower side of the head of studs and the edge distance of the stud from the steel flange. Detailing rules for solid slabs are illustrated in Table 4.20. For other cases reference is made to the literature.

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5 Design of connections and joints

Abstract. Connections and joints constitute a very important part of structural steelwork, to be met during its fabrication and erection. This chapter introduces the basis of design for mechanical and welded connections and joints. It gives the specifications of the different types of bolts and accessories, the geometrical properties for bolted assemblies and the installation of bolts. It then introduces the main categories of bolted connections, whether bearing or slip resistant at SLS or ULS when subjected to shear forces and non-preloaded or preloaded when subjected to tension forces. For these types of bolted connections and for connections with pins it gives the Eurocode 3 provisions to determine the design resistances and preforming the relevant checks. It presents welding connections, providing technological information on welding methods, residual stresses and welding deformations. It defines the types and geometric properties of welds giving the design resistances of fillet, butt and plug welds. It then gives the design of joints subjected to shear forces, whether long lap joints, or splices of members, providing elastic and plastic methods for groups of fasteners. Subsequently it introduces the T-stub as the basic element of joints subjected to tension or compression forces. It then provides the classification of joints, whether by stiffness or by strength. It finally presents design methods for typical joints, such as beam-to-column joints, welded or bolted, column bases, or hollow section joints.

5.1 Introduction

Connections in steel structures have the purpose:

a) To connect different steel members and sheets to complete structures.
b) To form cross-sections and members from the final steel making products, such as cross-sections from steel plates, built-up members from rolled or welded sections and plates etc.
c) To splice members which are delivered in partial length due to transportation restrains and form members of full length.

Depending on the type of the connecting media, connections are distinguished in:

a) Connections with mechanical connecting media such as bolts, pins, rivets, screws etc.
b) Welded connections.

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Rivets were the most popular connection media from the middle of the 19th to the middle of the 20th century. They usually required four workers for their installation; one to warm the rivet, two to hammer them alternately after installation in order not to cool and one to hold against. Due to high labor costs rivets were gradually substituted from bolts after 1950, although bolts are as material more expensive and unlike rivets they do not completely fill the holes so that an empty space remains that leads to additional slip deformations. On the other side, bolts may unlike rivets transfer tension forces and bolted constructions may be easily assembled, de-assembled and removed. Other types of mechanical fasteners like blind rivets, self-tapping screws or cartridge fired pins have been developed in connections for thin walled elements and structures.

Welding of metals is as old as forging and has been used unchanged for centuries. The pieces were forged to a conical shape, heated to white heat and fused by forging. However, the way for industrial application opened in 1881 when N. Benardos from Russia connected a carbon rod to an electric supply that produced an electric arc between the electrode and a work piece. The arc melted both the work piece and a stick from the same metal and fused the piece together [5.1]. The method found wider application by the end of the 19th century, while at the same time gas welding developed where the flame was produced by an oxygen acetylene torch. The extensive application of welding started later during the 1st world war in shipbuilding and later in construction. Welding is currently used extensively in the construction sector, preferably for shop connections, while bolting is the preferred method for field connections.

This chapter presents design of connections and joints to Eurocode 3, part 1.8 [5.2]. Rules for welded connections are valid for material thicknesses 4 mm and over. Figure 5.1 illustrates the distinction between the following two terms. Connections join two structural elements, like the welded beam-to-column connection and the bolted connection for a beam splice, and may be regarded separately from the influence of other adjacent members. A joint refers to the connection area for the ensemble of all elements whose axes converge, or nearly converge, to a geometrical point so that they mutually influence each other. Examples of connections and

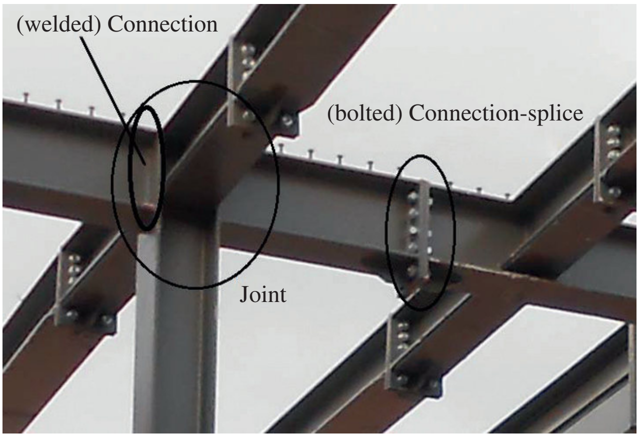


Fig. 5.1. Examples of connections and joints

joints are illustrated in Figure 5.1. In this figure four beams connect separately to one column to form a joint that its behavior depends on the type of each individual beam-column connection, as well as the loading conditions at the ends of the joining members.

Different types of connection media attract different loading portions due to different stiffness and should not be used to share loads in the same joint. For example, bolts should not be used together with welds in the same connection, since welds are stiffer and would attract most part of the load. For the same reason non-preloaded bolts should not share loads in the same joint with preloaded bolts which are stiffer. Eurocode 3 [5.2] allows the combination between welds and preloaded bolts provided that the bolts are preloaded after the welding is complete, but this should be done only in exceptional cases where no alternative exists.

5.2 Bolted connections

5.2.1 Bolts and accessories

Bolts are composed of the head and the shank which has an unthreaded and a threaded part. Together with nuts, washers and lock washers they form bolting assemblies that shall be supplied by one manufacturer who is responsible for the function of the assembly (Figure 5.2). Metric bolts to ISO 898-1 [5.3] are made of carbon steel or alloy steel, have ISO metric screw thread, hexagonal head and are designated by the letter M, meaning metric, and a figure equal to their shaft diameter in mm. As an example, M20 is a metric bolt to ISO 898-1 with 20 mm diameter. There exist bolts with round countersunk head with internal driving feature which may have reduced load ability, whereas such with left-hand thread must be appropriately marked.

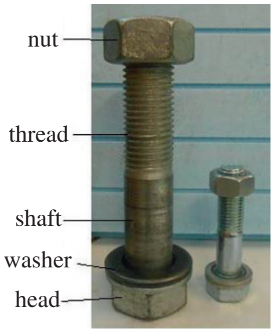


Fig. 5.2. Bolts and accessories

Bolts to ISO 898-1 are divided in property classes with specified mechanical properties, physical properties and chemical composition. The symbol of the class consists of two numbers separated by a dot, where the first number indicates 1/100 of the nominal tensile strength in MPa and the second 10 times the ratio between the nominal "yield" strength to the nominal tensile strength. Table 5.1 presents nominal values of the yield strength, tensile strength and fracture elongation for the various classes. Distinction is made between normal and high strength bolts, where only high strength bolts may be used as preloaded bolts in slip-resistant connections. High strength bolts are of two types with designations HR (Haute Résistance) and HV (Hochfest Vorgespannt) indicating their French and German origin which differ mainly in the head and nut dimensions. Their properties along with their application are specified in the EN 14399 family of specifications, which is divided in 10 parts [5.4] to [5.13]. It should be mentioned that nominal values for strength are understood as minimum guaranteed values and not fractile values with a specified allowance for non-compliance.

Table 5.1. Mechanical properties of bolts [5.3]

Bolt class	**4.6**	4.8	**5.6**	5.8	**6.8**	**8.8**	**10.9**	12.9
Designation	Normal bolts					High strength bolts		
f_y [MPa]	240	320	300	400	480	640	900	1080
f_u [Mpa]	400	400	500	500	600	800	1000	1200
ε_u [%]	22	24	20	22	20	12	9	8
Remark	with *bold* are indicated classes covered by the rules of EN 1993-1-8 [5.2]							

Tolerance of form and position are important for use of fasteners. Accordingly, product grades A, B and C are introduced that refer to tolerance levels, where grade A is the most precise and grade C the least precise. Dimensions for bolts and accessories are defined in various standards such as ISO, EN or DIN. Table 5.2 gives a limited overview of specifications for bolts, nuts and washers, while Table 5.3 provides dimensions for high strength bolts and their accessories.

Table 5.2. Specifications for bolts and accessories

Bolts		*Nuts*		*Washers*	
Specification	*Class/Type*	*Specification*	*Type*	*Specification*	*Type*
DIN 7968	5.6	EN ISO 4032	A, B	DIN 434	
DIN 7969	4.6	EN ISO 4034	C	DIN 435	
DIN 7990	4.6 5.6			DIN 7989	
EN 14399-3	8.8 & 10.9 HR	EN 14399-3	HR	EN 14399-5 & 6	HR
EN 14399-4	8.8 & 10.9 HV	EN 14399-4	HV	EN 14399-5 & 6	HV
EN 14399-8	8.8 & 10.9 HV fit	EN 14399-8	HV	EN 14399-5 & 6	HV

Table 5.3. Dimensions for high strength bolts and accessories [mm]

Specification	EN 14399-3 & 4							
Bolt size	M12	M16	M20	M22	M24	M27	M30	M36
d nominal diameter of unthreaded shank	12	16	20	22	24	27	30	36
A_s stress area [mm^2]	84.3	157	245	303	353	459	561	817
Thickness of head max. HV	8.45	10.75	13.90	14.90	15.90	17.90	20.05	24.05
Thickness of head max. HR	7.95	10.75	13.40	14.90	15.90	17.90	19.75	23.55
Fit bolts	EN 14399-8							
d nominal diameter of unthreaded shank	13	17	21	23	25	28	31	37
Nuts	EN 14399-3 & 4							
Width across corners	23.91	29.56	35.03	39.55	45.20	50.85	55.37	66.44
Width across flats max.	22.00	27.00	32.00	36.00	41.00	46.00	50.00	60.00
Thickness max. HV	10.00	13.00	16.00	18.00	20.00	22.00	24.00	29.00
Thickness max. HR	10.80	14.80	18.00	19.40	21.50	23.80	25.60	31.00
Washers								
Diameter HV	20.10	24.90	29.50	33.30	39.55	45.20	49.00	58.80
Diameter HR	20.10	24.90	29.50	33.30	38.00	42.80	46.60	55.90
Thickness for HV, HR in mm	3	4	4	4	4	5	5	6

Fit bolts are used when slip between the connected parts shall be limited to a minimum without preloading the bolt. In fit bolts the diameter of the threaded portion of the shank is 1 mm smaller than the diameter of the unthreaded portion. This allows a very small clearance between bolt shank and bolt hole that is for fit bolts 0.3 mm. The thread of a fit bolt shall not pass through the shear plane(s) and shall not extend beyond 1/3 of the thickness of the plate that is in contact with the nut.

The length of bolts depends on the thickness of the connected parts and the required bolt end protrusion beyond the nut face as specified in EN 1090-2 [5.22]. Washers are not necessary for non-preloaded bolts in normal holes, unless for inclined connection surfaces with inclination angles larger than 2^0 or 3^0 for bolts larger or correspondingly smaller than 20 mm (taper washers). For pre-loaded 8.8 or 10.9 bolts, one or correspondingly two washers are needed for better load distribution in the connection elements. Washers are also required for oversize round holes, long slotted holes and single lap joints with only one bolt or one row of bolts.

5.2.2 Hole clearances and bolting assemblies

Bolt holes are larger than bolt shanks in order to allow an easy placement of the bolt. The hole diameter is equal to:

$$d_0 = d + c \tag{5.1}$$

where:

d_0 = hole diameter
d = bolt diameter
c = nominal hole clearance

According to the clearance c, four types of holes as specified by EN 1090-2 [5.22] are distinguished.

- *Normal round holes*
  - $c = 1$ mm for bolts M12 and M14
  - $c = 2$ mm for bolts M16 to M24
  - $c = 3$ mm for bolts $\geqslant$ M27

The clearance for M12 and M14 bolts may be increased to 2 mm under the conditions set up in section 5.5.

- *Oversize round holes*
  - $c = 3$ mm for bolts M12
  - $c = 4$ mm for bolts M14 to M22
  - $c = 6$ mm for bolts M24
  - $c = 8$ mm for bolts $\geqslant$ M27
- *Short slotted holes* (*Figure* 5.3)
  Clearance in axis y: like in normal round holes
  Clearance in axis x:
  - $c = 4$ mm for bolts M12 and M14
  - $c = 6$ mm for bolts M16 to M22
  - $c = 8$ mm for bolts M24
  - $c = 10$ mm for bolts $\geqslant$ M27

- *Long slotted holes* (*Figure* 5.3)
  Clearance in axis y: like in normal round holes
  Clearance in axis x: it is $d_0 = 2.5 \cdot d$

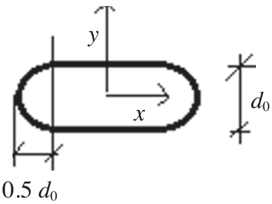


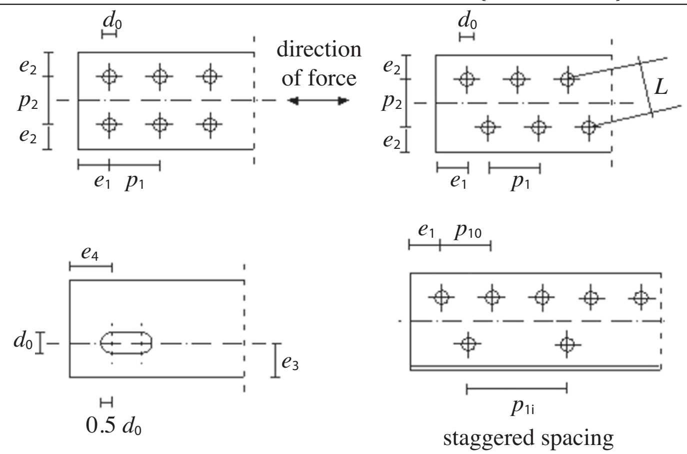
Fig. 5.3. Geometry of slotted holes

For fit bolts the nominal diameter of hole is equal to the shank diameter of the bolt.

There exist upper and lower limits concerning spacing between bolts as well as end and edge distances. Minimum values are defined in order to not reduce excessively the bearing capacity. Maximum values are defined to avoid local buckling between bolts for members in compression and prevent corrosion for exposed tension members. Spacing is defined by the letter p, distances from the ends by the letter e. Symbols receive the index 1 if parallel to the force or 2 if perpendicular to the force. Table 5.4 presents the limitations as provided by EN 1993-1-8 [5.2]

Table 5.4. Limitations for bolt spacing and distances. Steels with normal atmospheric corrosion protection

Distances e_1 *end distance* e_2 *edge distance*		*Spacing* p_1 *parallel to the force* p_2 *perpendicular to the force*	
min e_1 min e_2	$1.2 \cdot d_0$	min p_1	$2.2 \cdot d_0$
		min p_2	$2.4 \cdot d_0$ or $1.2 \cdot d_0$ for staggered
min e_3	$1.5 \cdot d_0$	min L	$2.4 \cdot d_0$
min e_4	$1.5 \cdot d_0$		
max e_1 max e_2	Steel exposed to weather or corrosive influences 40 mm + 4 t otherwise not applicable	max p_1 max p_2	*Compression members* min{14 t; 200 mm} *Tension members*: Outer rows min{14 t; 200 mm} Inner rows min{28 t; 400 mm}



Recommended positions of holes for rolled sections are provided in the relevant cross-sections tables. These positions do not always comply with the minimum edge distances of Table 5.4 as required by the Code.

5.2.3 Installation of bolts

Execution of bolting for non-preloaded bolts is performed in three steps:

- Holing by drilling, cutting or punching, the latter under condition for high execution classes, see EN 1090-2.
- Introduction of bolts, nuts and washer, if applicable.
- Tightening of bolts in a manner to achieve a uniform snug-tight position

For preloaded-bolts in slip resistant connections two additional steps are required:

- Preparation of contact surfaces to achieve the required slip factor
- Use of two washers from both sides of the plates for bolts 10.9 or one for 8.8
- Tightening of bolts to achieve the required preloading force.

The preloading force $F_{p,c}$ is determined from equation (5.2). Its numerical values are given in Table 5.5.

$$F_{p,c} = 0.7 \cdot f_{ub} \cdot A_s \tag{5.2}$$

where:

f_{ub} is the nominal ultimate strength of the bolt, Table 5.1 and
A_s is the stress area of the bolt, Table 5.3.

Tightening may be performed according to three methods as described in the following. A fourth method exists for HRC bolts to [5.13]. For all methods tightening should be carried out progressively, possibly in more cycles to achieve uniform preloading.

Torque method
The bolts are tightened by a torque wrench to the torque $M_r = k_m \cdot d \cdot F_{p,C}$, where d is the bolt diameter and k_m a factor declared by the manufacturer. Tightening is carried out in two steps, first at 75% of the torque and then to 110% of the torque. M_r is a reference value for the torque for both the torque and the combined method.

Combined method
The bolts are tightened in two steps. In the first step by a torque wrench to 75% of the torque M_r and after completion of it for all bolts by turning the nut by a certain angle ranging between 60^0 and 120^0 in dependence on the relation between the thickness of the connected parts and the bolt diameter.

Table 5.5. Preloading force $F_{p,c}$ [kN]

	M12	M16	M20	M22	M24	M27	M30	M36
Bolts of class 8.8	47	88	137	170	198	257	314	458
Bolts of class 10.9	59	110	172	212	247	321	393	572

Direct tension indicator method
This method relies on the use of appropriate compressible washers that indicate achievement of the preloading force and is described in EN 14399-9 [5.12].

Inspection of correct execution of the works ranges from visual checking to checking the preloading force for a percentage of bolts, in dependence on whether the bolts are preloaded or not, the execution class and the tightening method.

5.2.4 Categories and resistance of bolted connections

5.2.4.1 General

According to EN 1993-1-8, five categories [5.2] of bolted connections, A, B, C, D, or E, are distinguished (Table 5.6). The first three refer to connections transferring shear forces perpendicular to the bolt axis, the last two for connections transferring tension forces parallel to the bolt axis. In category A belong non-preloaded shear connections of bearing type; in category B preloaded shear connections that are slip resistant at SLS; in category C preloaded shear connections that are slip resistant at ULS. In category D belong non-preloaded connections transferring tension forces, while in category E preloaded connections transferring tension forces. The types of connections and the required checks are summarized in Table 5.6. Further details are given below.

Table 5.6. Categories of bolted connections and design checks [5.2]

Category	*Design checks*	*Field of application*
	Shear forces	
A Bearing connections	$F_{v,Ed} \leqslant F_{v,Rd}$ shear capacity check $F_{v,Ed} \leqslant F_{b,Rd}$ bearing capacity check	Non-preloaded All bolt classes
B Slip-resistant connections at SLS	*Forces at SLS* $F_{v,Ed,ser} \leqslant F_{s,Rd,ser}$ slip check *Forces at ULS* $F_{v,Ed} \leqslant F_{v,Rd}$ shear capacity check $F_{v,Ed} \leqslant F_{b,Rd}$ bearing capacity check	Preloaded Bolt classes 8.8 and 10.9
C Slip-resistant connections at ULS	$F_{v,Ed} \leqslant F_{s,Rd}$ slip check $F_{v,Ed} \leqslant F_{b,Rd}$ bearing capacity check	Preloaded Bolt classes 8.8 and 10.9
	Tension forces	
D Non-preloaded connections	$F_{t,Ed} \leqslant F_{t,Rd}$ tension check $F_{t,Ed} \leqslant B_{p,Rd}$ punching check	All bolt classes
E Preloaded connections	$F_{t,Ed} \leqslant F_{t,Rd}$ slip under tension check $F_{t,Ed} \leqslant B_{p,Rd}$ punching check	Bolt classes 8.8 and 10.9

5.2.4.2 Resistance to shear forces

Bearing connections, category A

Category A connections are those where non-preloaded bolts are subjected to forces perpendicular to their axis. In order to transfer the applied forces slip must take place in order the connected parts come in contact with the bolt shaft. Accordingly, these connections may be used to resist loads only for normal rounded holes or short slotted holes transverse to the slot. Load transfer is through bearing stresses in the connected parts and shear stresses in the bolt shaft (Figure 5.4a). Accordingly, the modes of failure are due to shear failure of the bolts or bearing failure of the connected parts (Figure 5.4b, c). The relevant design resistances are summarized in Table 5.7. It should be mentioned that bearing stresses are not uniform in the elastic range across the section but concentrate at hole edges. However, these stress concentrations alleviate as loading progresses due to yielding so that stresses are supposed to be uniform at ULS.

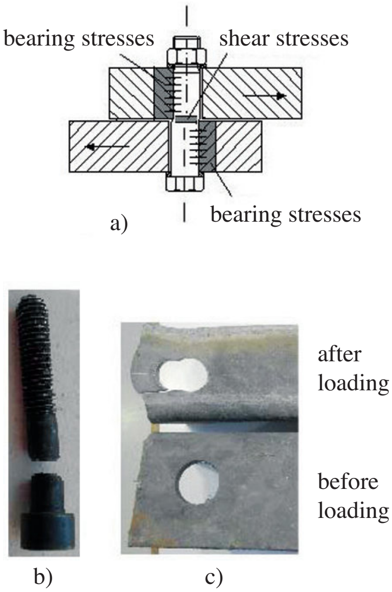


Fig. 5.4. Category A connections: a) Stresses, b) shear failure c) bearing failure

Slip resistant connections category B or C

When the bolts are preloaded, shear forces are transferred through friction between the parts (Figure 5.5). As widely known, friction forces depend on the values of the friction coefficient and the transverse force. Accordingly, slip resistant connections need preparation of the contact surfaces in order to ensure a certain friction coefficient and preloading of bolts to create the trans-

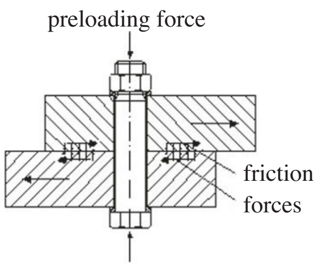


Fig. 5.5. Preloading and friction forces in slip resistant connections

Table 5.7. Design shear and bearing resistances of bolts [5.2]

Shear resistance of a bolt

The shear plane passes through the unthreaded portion of the bolt:

$$F_{v,Rd} = n \cdot \frac{0.6 \cdot f_{ub} \cdot A}{\gamma_{M2}} \tag{5.3}$$

The shear plane passes through the threaded portion of the bolt:

$$F_{v,Rd} = n \cdot \frac{0.6 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \text{ bolt classes 4.6, 5.6 and 8.8} \tag{5.4}$$

$$F_{v,Rd} = n \cdot \frac{0.5 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \text{ bolt classes 4.8, 5.8, 6.8 and 10.9} \tag{5.5}$$

Bearing resistance for normal holes

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} \tag{5.6}$$

Symbols

A = area of the unthreaded shaft A_s = tensile stress area of the bolt

$\alpha_b = \min\left\{\alpha_d, \frac{f_{ub}}{f_u}, 1.0\right\}$

In the direction of load transfer

$\alpha_d = \frac{e_1}{3d_0}$ for end bolts $\alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$ for inner bolts

Perpendicular to the direction of load transfer

$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7 \text{ or } 2.5\right\}$ for edge bolts $k_1 = \min\left\{1.4\frac{p_2}{d_0} - 1.7 \text{ or } 2.5\right\}$ for inner bolts

d, d_0 = bolt diameter, hole diameter.

f_u, f_{ub} = ultimate strength of connected plates and of bolts.

t = minimum thickness, or sum of thicknesses, of connected plates in every force direction. For countersunk bolts half of the depth of countersinking is subtracted.

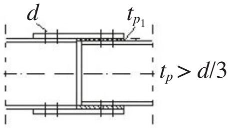
n = number of shear planes.

$\gamma_{M2} = 1.25$ partial safety factor.

For oversize holes the bearing resistance is multiplied by 0.8.

For slotted holes, where the force is parallel to the slot the bearing resistance is multiplied by 0.6.

The shear resistance $F_{v,Rd}$ of bolts transmitting forces through packings of total thickness t_p greater than $d/3$ should be reduced by the factor $\beta_p = \frac{9d}{8d + 3t_p} \leqslant 1$. For double shear connections with packings on both sides, t_p is the thickness of the thicker packing.



The bearing resistance in single lap joints with only one bolt row should be limited per bolt to $F_{b,Rd} \leqslant 1.5 \cdot f_u \cdot d \cdot t / \gamma_{M2}$

verse force. Before the applied shear forces exceed the friction, no slip occurs between the connected parts. However, when the shear forces increase to overcome friction, slip takes place, the bolt shaft comes in contact with the plates and the connection behaves like a bearing connection of category A. "Failure" is defined conventionally as the start of slip although the connection is able to resist higher forces beyond this state. That is the reason why the slip resistant connections have lower "resistance" compared to bearing type connections. Depending on the level of forces for which slip is avoided, slip resistant connections are distinguished in:

- Category B connections that are slip resistant to SLS forces
- Category C connections that are slip resistant to ULS forces

Slip resistant connections are used in structures subjected to dynamic loads, such as bridges, wind towers and generally when deformations shall be limited to a minimum. They must be used for plates with oversize or slotted holes. Table 5.8 gives design values for slip resistance of preloaded bolts.

Table 5.8. Design slip resistances of preloaded bolts [5.2]

	Slip resistance
	$$F_{s,Rd} = \frac{k_s \cdot \eta \cdot \mu}{\gamma_{M3}} \cdot F_{p,c} \quad (5.7)$$
	Symbols
$F_{p,c} =$	preloading force
$\eta =$	number of friction surfaces
$k_s =$	1.0 normal holes
	0.85 oversize holes or short slotted holes with the axis of the slot perpendicular to the force
	0.7 long slotted holes with the axis of the slot perpendicular to the force
	0.76 short slotted holes with the axis of the slot parallel to the force
	0.63 long slotted holes with the axis of the slot parallel to the force
$\mu =$	slip factor with following values
$\mu =$	0.5 category A friction surfaces (blasted surfaces to remove loose rust, not pitted)
	0.4 category B friction surfaces (spray-metallized with an Al or Zn based product or with alkali-zinc silicate paint, thickness 50-80 μm)
	0.3 category C friction surfaces (surfaces cleaned by wire-brushing or flame cutting, loose rust removed)
	0.2 category D friction surfaces (untreated as rolled surfaces)
$\gamma_{M3} =$	1.25 category C slip resistant connections
$\gamma_{M3} =$	1.10 category B slip resistant connections
	Bearing resistance (as in Table 5.7)

5.2.4.3 Behavior and resistance to tension

Forces parallel to the axis of the bolt produce tension in the bolts. Here two connection categories are distinguished:

- Category D connections with non-preloaded bolts
- Category E connections with preloaded bolts

Table 5.9. Design resistance to tension forces [5.2]

Tension resistance of a bolt

$$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \tag{5.8}$$

Punching shear resistance of the plate

$$B_{p,Rd} = \frac{0.6 \cdot \pi \cdot f_u \cdot d_m \cdot t_p}{\gamma_{M2}} \tag{5.9}$$

Symbols

$k_2 = 0.9$ in general
0.63 for countersunk bolts
f_u = ultimate strength of connected plates
f_{ub} = ultimate strength of bolts
d_m = the mean value between in- and circumscribed diameters of bolt head and bolt nut, whichever is smaller
t_p = thickness of washer
$\gamma_{M2} = 1.25$

Tension forces in non-preloaded bolts are transmitted through the shaft of the bolt so that the bolt fails at its weaker section of the thread. An additional failure mode is punching shear of the plates. The design resistance to tension forces is presented in Table 5.9.

In presence of tension forces in preloaded bolts, the preloading force is reduced and so does the slip resistance when shear and tension forces are combined.

In presence of combined shear and tension forces additional checks are required. In category D connections the shaft of the bolt is subjected to direct and shear stresses and must be checked accordingly. In category B or C connections, the preloading force is reduced and so does the resistance to slip. Table 5.10 presents the required checks for combined tension and shear forces.

The study of the behavior of non-preloaded and preloaded bolts to tension forces is essential since it affects strongly the fatigue resistance. In a connection between two plates with one bolt, the plates and the bolt may be represented by bilinear axial springs acting in parallel where the bolt spring is tension only, the plate spring compression only. The mechanical model and the spring properties are presented in Table 5.11. The plate spring is developed under the assumption that the force is distributed by 45^0.

After preloading, the bolt is subjected to a tension force F_p that elongates it, while the plates are subjected to an equal and opposite compression force $-F_p$ that contracts them. When an external force F_{Ed} is applied, the force is distributed between

Table 5.10. Design resistance to combined tension and shear [5.2]

Category D and E connections

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 \cdot F_{t,Rd}} \leqslant 1 \qquad (5.10)$$

Slip resistance to combined tension and shear forces

Category B connections

$$F_{s,Rd,ser} = \frac{k_s \cdot \eta \cdot \mu \cdot \left(F_{p,c} - 0.8F_{t,Ed,ser}\right)}{\gamma_{M3}} \qquad (5.11)$$

$\gamma_{M3} = 1.10$

Category C connections

$$F_{s,Rd} = \frac{k_s \cdot \eta \cdot \mu \cdot \left(F_{p,c} - 0.8F_{t,Ed}\right)}{\gamma_{M3}} \qquad (5.12)$$

$\gamma_{M3} = 1.25$

the bolt and the plate in analogy to their stiffness as follows:

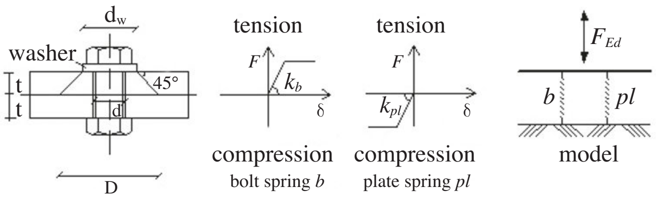
$$\text{Bolt:} \qquad F_b = \frac{k_b}{k_b + k_{pl}} \cdot F_{Ed} \qquad (5.13)$$

$$\text{Plates:} \qquad F_{pl} = \frac{k_{pl}}{k_b + k_{pl}} \cdot F_{Ed} \qquad (5.14)$$

where the spring stiffness is determined from Table 5.11.

The final forces in the bolt and the plate are determined as the sum between the forces due to preloading and the external force F_{Ed}. The forces in the bolt and the connected plates may be illustrated in the same diagram. The left part of Figure 5.6a) and b) shows the forces in the bolts and the plates when an external tension or corre-

Table 5.11. Mechanical model and spring properties of bolt and plate



Bolt		*Plate*	
Stiffness	Yield load	Stiffness	Yield load
$k_b = \frac{E \cdot A_s}{2 \cdot t}$ A_s stress area of bolt	$F_{t,Rd}$ tension resistance of bolt, Table 5.9	$k_{pl} = \frac{E \cdot A_{pl}}{2 \cdot t}$ $A_{pl} = \frac{\pi \cdot (D^2 - d^2)}{4}$ $D = d_w + t$	$F_{y,Rd} = A_{pl} \cdot f_y$ f_y yield strength of plate

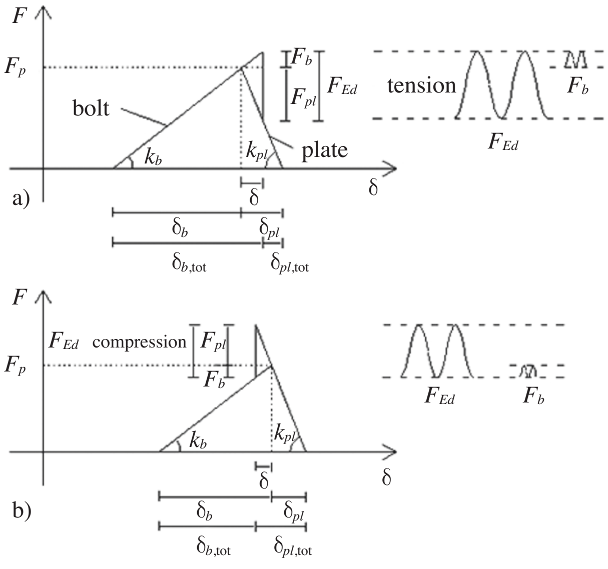


Fig. 5.6. Forces in preloaded bolts and in the connected plates due to a) tension forces or b) compression forces parallel to the bolt axis and forces in the bolts due to cyclic loading

spondingly compression force parallel to the bolt axis is applied. It may be seen that the bolt forces are much lower that the applied external force. The right part of Figure 5.6 shows the fluctuation of bolt forces for cyclic external loading. It may be seen that the bolt forces fluctuate much less than the external load, which has evidently important consequences on the fatigue behavior. For this reason, bolted connections subjected to dynamic cyclic loading parallel to the bolt axes should be preloaded.

5.3 Connections with pins

Pins are used whenever free rotation between the connected parts is required so that the connection behaves as a hinge as close as possible (Figure 5.7). Pin joints transfer therefore only forces and no moments. The ends of the connected members are provided with eye-bar plates through which run the pins. The eye-bars have round holes with clearance so that the pin can be easily inserted and possibly replaced. EN 1993-1-8 [5.2] assumes in its model a uniform stress distribution across the thickness of the eye-bars, which is simple because the pin rotation results in stress concentrations at the edges.

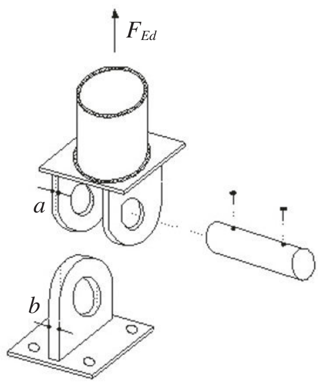


Fig. 5.7. Pin connection with eye-bars

The dimensions of the eye-bars may be determined by two alternative methods as illustrated in

Table 5.12. Eye-bar dimensions by two alternative methods [5.2]

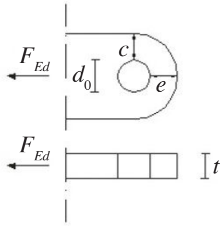
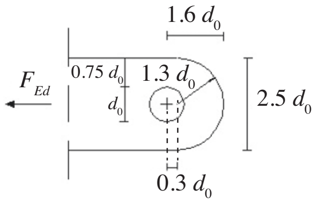
Specified thickness t	*Specified end and edge distances*
	
$e \geqslant \frac{F_{Ed} \cdot \gamma_{M0}}{2 \cdot t \cdot f_y} + \frac{2 \cdot d_0}{3}$ and $f \geqslant \frac{F_{Ed} \cdot \gamma_{M0}}{2 \cdot t \cdot f_y} + \frac{d_0}{3}$	$t \geqslant 0.7 \cdot \left[\frac{F_{Ed} \cdot \gamma_{M0}}{f_y}\right]^{1/2}$ and $d_0 \leqslant 2.5 \cdot t$

Table 5.12. A simple model treats the pin as a simply supported beam subjected to uniformly distributed loading from the internal eye-bar which has a span equal to the axial distance of the external eye-bars (Figure 5.8). The pins are checked, like bolts, to shear and bearing and in addition to bending and combined bending and shear as summarized in Table 5.13.

Table 5.13. Design checks for pins [5.2]

Check	*Design formula*
Shear of pin	$F_{v,Ed} \leqslant F_{v,Rd} = 0.6 \cdot A \cdot f_{up}/\gamma_{M2}$
Bearing of the plate and the pin	$F_{v,Ed} \leqslant F_{b,Rd} = 1.5 \cdot d \cdot t \cdot f_y/\gamma_{M2}$
Bearing of the plates and the pins for replaceable pins	$F_{v,Ed,ser} \leqslant F_{b,Rd,ser} = 0.6 \cdot d \cdot t \cdot f_y/\gamma_{M6,ser}$
Bending of pin	$M_{Ed} \leqslant M_{Rd} = 1.5 \cdot W_{el} \cdot f_{yp}/\gamma_{M0}$
Bending of pin for replaceable pins	$M_{Ed} \leqslant M_{Rd} = 0.8 \cdot W_{el} \cdot f_{yp}/\gamma_{M6,ser}$
Pin to combined bending and shear	$\left(\frac{M_{Ed}}{M_{Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leqslant 1$

f_{up} = ultimate strength of pin,
f_{yp} = yield strength of pin
f_y = yield strength of eye-bars
d = diameter of pin
t = thickness of eye-bars
$A = \frac{1}{4} \cdot \pi \cdot d^2$ = cross-section area of pin
$W_{el} = \frac{1}{32} \cdot \pi \cdot d^3$ = elastic modulus of pin cross-section
$\gamma_{M0} = 1.0 \quad \gamma_{M2} = 1.25 \quad \gamma_{M6,ser} = 1.0$
For three eye-bars, Figure 5.7:
$F_{v,Ed} = 0.5 F_{Ed} \quad M_{Ed} = \frac{F_{Ed}}{8} \cdot (b + 4c + 2a)$

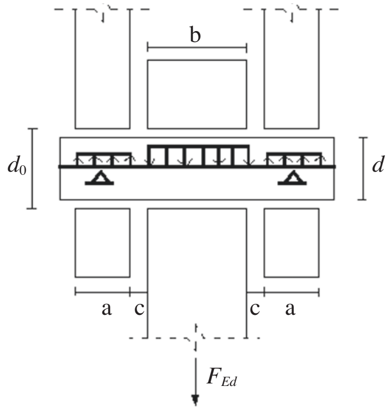


Fig. 5.8. Simplified model for design of pins

5.4 Welded connections

5.4.1 Welding methods

Welding in the construction sector is most usually performed by melting methods, where large temperatures develop that melt the surfaces of the parent metal in the welding area as well as an electrode and allow their fusion after solidification. High temperatures develop through an electric arc that forms between the parent metal and the electrode both of which are connected to a power source as shown in Figure 5.9.

The electrode is composed of a core steel wire and a flux covering. The steel wire of the core provides the melting material while the cover adds alloy elements that provide the slag and release gases during welding that stabilizes the arc and shields it from the air. This is due to the fact that an unprotected arc is not stable and hygroscopic, so that it absorbs gases, such as oxygen (O_2), hydrogen (H_2) or nitrogen (N_2), from the air that are harmful to the weld.

Oxygen reacts with iron (Fe) and releases gases CO_2 and SO_2 that when leaving the melt lead to pores in the weld. Oxygen may be bound with Si and Mn that are

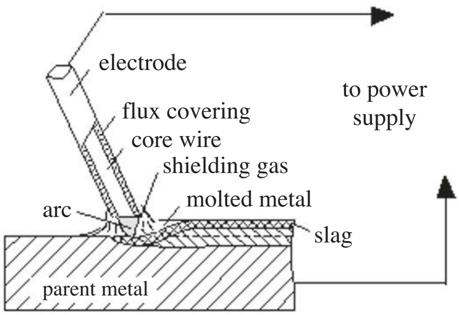


Fig. 5.9. Arc welding with an electrode (E)

added to the melt. Hydrogen comes from humidity in the covering, rust, paint, fats, water etc. It is very dangerous to the weld since it creates pores, micro- or macro-cracks, or brittleness. That is why the fusion surfaces must be clean, the environment and the electrodes dry, the cooling speed low and the slag that rests on the weld removed.

The electrodes may be acid, basic or rutile. Basic electrodes are the best since they do not provide oxygen in the melt. However, they are the most difficult to weld and hygroscopic so they must be dried before use. During welding the metallurgical composition in the weld area changes since the regions that solidify first have small, those that solidify last high content on alloy elements. The overlying slag, which is removed later, reduces the cooling speed and improves the properties in the weld region. The area affected by high temperature and metallurgical changes during welding includes the weld itself and a zone around it called the heat affected zone (HAZ).

Other welding methods in construction include metal active gas welding (MAG), tungsten inert gas welding (TIG) and submerged arc welding (SAW), Figure 5.10. In MAG welding, the wire is unwind from a spool and continuously supplied so that the welding operation must not be stopped in order to substitute the electrode. The weld is protected by an active gas, CO_2, which is supplied through a nozzle. The difference between MAG and TIG welding is that in TIG welding the gas is Argon, an inert gas, the electrode is from tungsten, not from alloy steel, and a second wire is used.

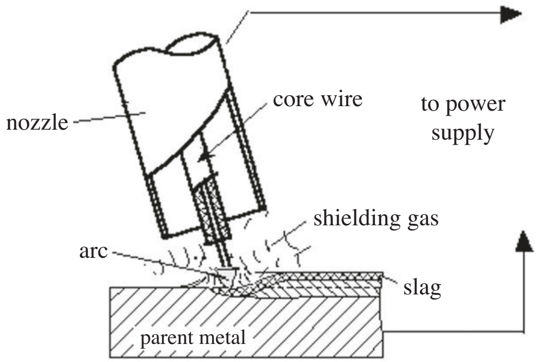


Fig. 5.10. Metal active gas welding (MAG)

Submerged arc welding provides high quality welds since the arc is covered by the powder and does not come in contact with the atmosphere, Figure 5.11. The powder melts and influences the melt through the reaction with the melted steel, the formation of the weld surface and the cooling speed. The method is very efficient for automatic welds of great length. It allows the execution of even thick welds in one run, unless the weld area is too large and there is danger to increase the size of the grain where more runs are needed. The method requires the use of a backing bar

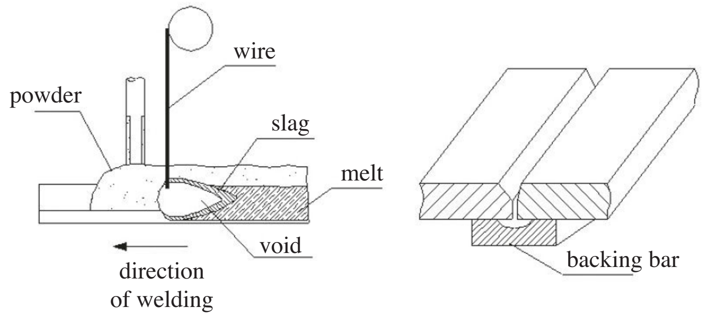


Fig. 5.11. Submerged arc welding (SAW)

from Cu, ceramic or other material for the protection of the root. Otherwise the root must be welded by another method, such as MAG.

For further reading reference is made to the literature [5.23] to [5.29].

5.4.2 Types and geometric properties of welds

It is reminded that the rules for welded connections to EN 1993-1-8 are valid for material thicknesses no smaller than 4 mm. Depending on the surface preparation, welds are distinguished in butt welds, fillet welds and plug welds, Table 5.14. Butt welds require appropriate edge preparation depending on the thickness of the connected parts. Furthermore, butt welds are distinguished in full and partial penetration welds.

The thickness of full penetration butt welds is considered equal to the thickness of the thinner connected plate. For transverse splices of plates of different thickness, the thicker plate must be tapered in thickness with a slope $\leqslant 1/4$. The throat thickness of partial penetration butt welds is not taken greater than the penetration thickness.

In partial penetration T-butt joints where the butt weld is reinforced by superimposed filled welds, Figure 5.12, the throat thickness may be considered equal to the plate thickness t, provided that following conditions apply:

$$a_1 + a_2 \geqslant t \text{ and } c \leqslant \min\{t/5, 3\text{ mm}\} \tag{5.15}$$

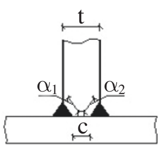


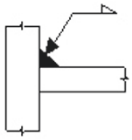
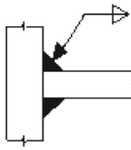
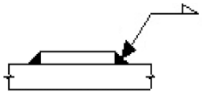
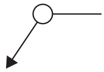
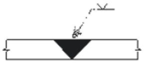
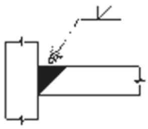
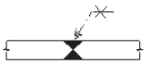
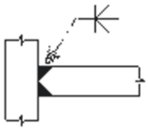
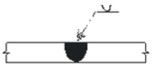
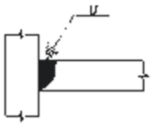
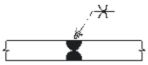
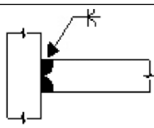
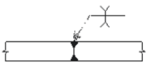
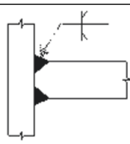
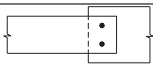
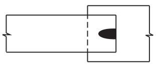
Fig. 5.12. Geometric properties of a T-butt joint

Partial penetration butt welds should not be used for the transfer of forces or moments perpendicular to the direction of the welds.

The throat thickness, a, of fillet welds is the height of the inscribed triangle within the weld, Figure 5.13. This applies also for deep penetration fillet welds, Figure 5.13 right, unless it is proved at site that the additional penetration can be consistently achieved in which case it is added to the throat. For the throat thickness of fillet welds that are covered by the rules of EN 1993-1-8 it shall apply $a \geqslant 3$ mm.

In inclined fillet welded T-joints the angle, ϕ, between plates shall be within the limits indicated by (5.16). For angles $\phi < 60^0$ the weld shall be considered as partial

Table 5.14. Types and symbols for welds

Type		*Simple*	*Double*
Fillet welds	T-joints		
	Lap joints		
		 Symbols for on-site execution of welds	 Symbol for all round welds
		Type of Groove	
Full penetration butt welds	single V - single Bevel		
	double V - double Bevel		
	single U - single J		
	double U - double J		
Butt welds	Partial penetration		
Plug welds	Lap joints	 	

penetration butt weld, while for $\phi > 120^0$ its resistance, if taken into account, shall be determined by testing.

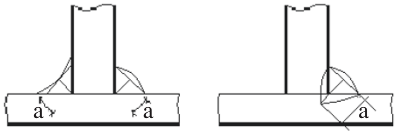


Fig. 5.13. Throat thickness of fillet welds and deep penetration fillet welds

$$60^0 \leqslant \phi \leqslant 120^0 \quad (5.16)$$

Fillet welds in lap joints shall have a minimum length of 6 times the throat thickness or 40 mm in order to carry forces. They shall be returned continuously around the corner at a minimum length indicated in Figure 5.14.

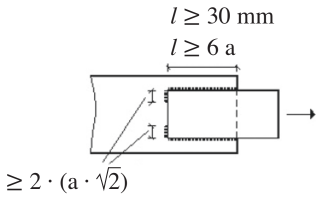


Fig. 5.14. Length of fillet welds in lap joints

Fillet welds may be intermittent. However, they must have continuous ends at a length L_{we}. Their geometric properties concerning the clear length of gaps and the required length of the continuous end are illustrated in Figure 5.15. Intermittent butt welds are not allowed.

$$L_{we} \geqslant \max\{0.75b; 0.75b_1\} \qquad L_1 \leqslant \min\{16t; 16t_1; 200\,\text{mm}\}$$
$$L_2 \leqslant \min\{12t; 12t_1; 0.25b; 200\,\text{mm}\}$$

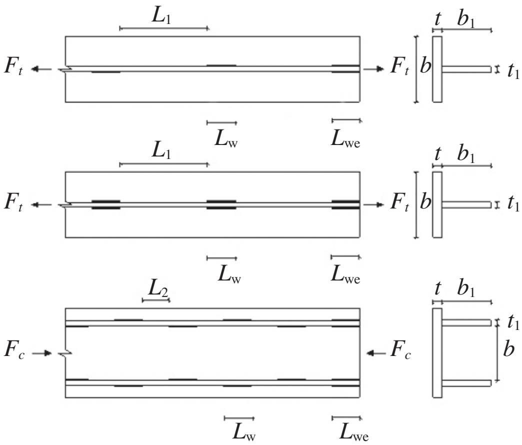


Fig. 5.15. Geometric requirements for intermittent fillet welds

Plug welds may be used:

- to transfer shear forces, not externally applied tension forces
- to prevent local buckling of lapped parts and
- to connect the components of built-up members

Plug welds are circular or elliptical with a rounded radius not less that the thickness t of the part containing them. Their minimum length dimension, D or b, should be at least 8 mm larger that the thickness t. The weld thickness, a, shall be within the limit indicated in Figure 5.16.

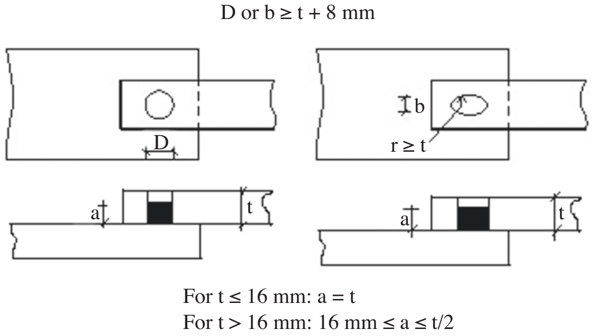


Fig. 5.16. Geometric requirements for plug welds

5.4.3 Design of welds

5.4.3.1 Fillet welds

Fillet welds may be designed by two methods, the directional method or the simplified method [5.2]. In the directional method direct and shear stresses in the plane perpendicular and transverse to the plane of its throat are considered as follows, Figure 5.17:

- $\sigma_\perp$ and $\sigma_{//}$ are the direct stresses that are perpendicular to the throat and correspondingly parallel to the axis of the weld.
- $\tau_\perp$ and $\tau_{//}$ are the shear stresses in the plane of the throat that are perpendicular and parallel to the axis of the weld correspondingly.

A force F_{Ed} acting on the centroid of the weld, Figure 5.17a, may be decomposed in its components $F_\perp$ and $F_{//}$ perpendicular and parallel to the axis of the weld which provoke following stresses:

$$F\perp \rightarrow \sigma\perp = \tau\perp = \frac{F_\perp}{A_w} \cdot \frac{\sqrt{2}}{2} = \frac{F_\perp}{2 \cdot a \cdot l_w} \cdot \frac{\sqrt{2}}{2} \tag{5.17}$$

$$F_{//} \rightarrow \tau_{//} = \frac{F_{//}}{A_w} = \frac{F_{//}}{2 \cdot a \cdot l_w} \tag{5.18}$$

A moment M_{Ed} in the plane of one connecting plate, Figure 5.17b, is provoking following stresses:

$$M_{Ed} \rightarrow \sigma_\perp = \tau_\perp = \frac{M_{Ed}}{W_{el,w}} \cdot \frac{\sqrt{2}}{2} = \frac{M_{Ed}}{2 \cdot a \cdot l_w^2/6} \cdot \frac{\sqrt{2}}{2} \tag{5.19}$$

For an eccentric acting force F_{Ed} with eccentricity e in relation to the weld's centroid, the resulting moment would be equal to $M_{Ed} = F_{Ed} \cdot e$ and the resulting stresses perpendicular to the weld would be calculated by adding the individual contributions from equations (5.17) and (5.19).

The design expressions for the fillet weld may be written as:

$$\sqrt{\sigma_\perp^2 + 3 \cdot (\tau_\perp^2 + \tau_{//}^2)} \leqslant \frac{f_u}{\beta_w \cdot \gamma_{M2}} \tag{5.20}$$

and

$$\sigma_\perp \leqslant \frac{f_u}{\gamma_{M2}} \tag{5.21}$$

where:

f_u is the smaller ultimate tensile strength of the plates joined,
β_w is an experimentally defined correlation factor depending on the steel grade, Table 5.15,
$\gamma_{M2} = 1.25$ is the partial safety factor for welds.

Table 5.15. Correlation factor β_w for welds

steel grade	S235	S275	S355	>S420
β_w	0.8	0.85	0.9	1.0

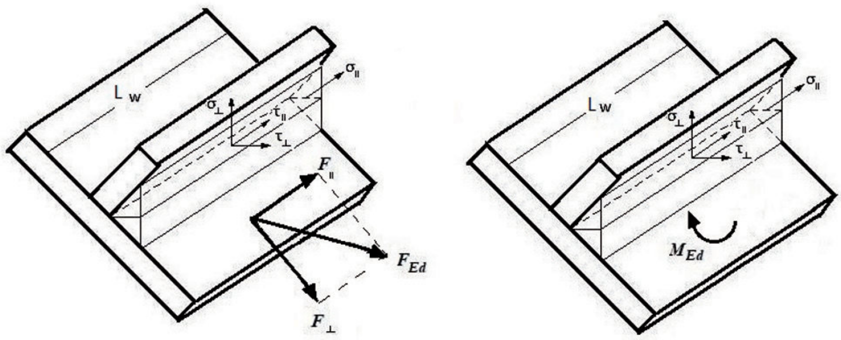


Fig. 5.17. Direct and shear stresses in fillet welds: a) Resolution of a concentric force in components parallel and transverse to the longitudinal axis, b) in-plane moment

The simplified method is an alternative to the directional method. In this method the maximum design force per unit length shall be limited by the corresponding resisting force per unit length. The relevant design criterion writes:

$$F_{w,Ed} \leqslant F_{w,Rd} \tag{5.22}$$

where:

$F_{w,Ed}$ is the maximum design weld force per unit length and
$F_{w,Rd}$ is the design weld resistance per unit length.

Independent from the force orientation, the design weld resistance is determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a \tag{5.23}$$

where:

a is the, throat, thickness of the weld
$f_{vw,d}$ is the design shear strength of the weld, determined from

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} \tag{5.24}$$

f_u, β_w and γ_{M2} are parameters as in the directional method.

For a weld subjected to a concentric force, Figure 5.17a, the application of the design expression (5.22) is straightforward since its left side is the, inclined, acting force divided by the length.

For a weld subjected to an eccentric force, i.e. simultaneously to a concentric force and an in-plane moment, Figure 5.17a and 5.17b, the design procedure is as follows:

- The force is resolved in its components parallel and perpendicular to the weld which create uniform force distributions per unit length:

$$F_{w//Ed} = \frac{F_{//}}{l_w} \quad \text{and} \quad F_{w\perp Ed} = \frac{F_{\perp}}{l_w} \quad \text{correspondingly.}$$

- The moment creates a triangular force distribution per unit length perpendicular to the weld with a maximum value $F_{M,w\perp Ed} = \frac{M_{Ed}}{l_w^2/6}$
- The maximal value of the total force per unit length, left side of equation (5.22), is calculated from

$$F_{w,Ed} = \sqrt{F_{w//Ed}^2 + (F_{w\perp Ed} + F_{M,w\perp Ed})^2}.$$

Obviously, for intermittent fillet welds the design force per unit length $F_{w,Ed}$ shall be determined excluding the un-welded length from the length of the weld.

5.4.3.2 Butt welds

The design of full penetration butt welds is covered by the design of the connected parts, since butt welds, properly executed, are supposed to be stronger that the connected elements.

Partial penetration butt welds have throat thickness smaller than the thickness of the connected parts. Accordingly, they are weaker than the connected parts and must be checked independently following the same rules as for fillet welds.

5.4.3.3 Plug welds

The design resistance of a plug weld is determined from:

$$F_{w,Rd} = f_{vw,d} \cdot A_w \tag{5.25}$$

where:

$f_{vw,d}$ is the design shear strength of the weld, eq. (5.24)
A_w is the design throat area, taken equal to the hole area.

5.4.4 Residual stresses

During welding, hot material is introduced in the gap between the two connecting parts (Figure 5.18). In the cooling phase this hot material would shrink if it would be allowed to freely deform, however it cannot do this since it is constrained by the surrounding cool material. Consequently, tension stresses develop in the weld which must be equilibrated by correspondent compression stresses in the connecting part. Figure 5.18 shows two pieces that are welded by a butt weld. The constraint introduced by the cool material is indicated by the end supports that do not allow any expansion-contraction of the pieces. Since the developing stresses are not in the elastic region, the two connecting pieces are no more stress free after the welding operation but develop tension and compression residual stresses. The part that cools last develops tensile residual stresses, nearly equal to the yield strength, while the other parts compression residual stresses. The distribution of residual stresses can, in general, be determined from force and moment equilibrium, due to the fact that residual stresses are self-equilibrating stresses.

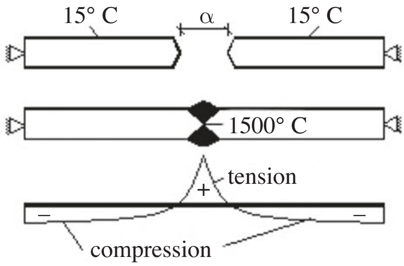


Fig. 5.18. Creation of residual welding stresses

Residual stresses constitute structural imperfections that develop during temperature changes in general and are therefore present not only in welded sections but also in rolled sections. Shapes of residual stresses for different sections are illustrated in Figure 5.19. They are similar for welded or rolled sections. The only difference is that in rolled sections the magnitude of residual stresses is less, i.e. tension stresses are a little smaller than the yield strength. Like in welded sections, in rolled sections tensile stresses appear at the web-flange junction that cool last, due to the fact that the surface to mass ratio in this region is lower than this ratio in the middle of the walls or the at the profile edges. The distribution of residual stresses for angles shown in Figure 5.19 indicates that tension stresses develop at the junction of the two legs where mass concentrates or a weld is present and that force and moment equilibrium conditions are satisfied separately at each leg.

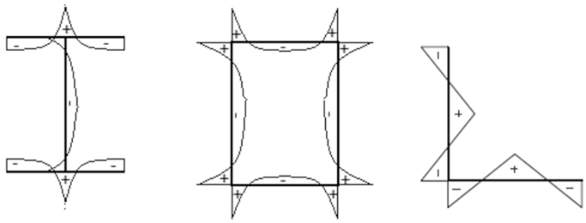


Fig. 5.19. Distribution of residual stresses, not in scale, for *I*-, box and angle sections (+ tension, - compression). Residual stresses act in longitudinal direction of the profile

5.4.5 Welding deformations

In addition to residual stresses, welding results in permanent deformations in members due to shrinkage of the welds. These deformations are distinguished in:

- shrinkage in longitudinal and transverse direction,
- angular distortion and
- bow deformation.

The amount of longitudinal shrinkage, Figure 5.20, depends on the ratio $\alpha = A/A_w$ (A is the cross-section area of the member and A_w the area of the weld). It increases with increasing weld area and may be calculated according to Table 5.16. Accordingly, the initial length of the member before welding should be a little larger than its nominal length, if necessary.

Table 5.16. Shortening of member

Formula	$\Delta l = \varepsilon \cdot l$					
$\alpha = \frac{A}{A_w}$	> 200	> 150	80	65	50	> 30
ε ‰	0	0.1	0.3	0.65	1.0	1.0-1.25
Notation	See Figure 5.20					

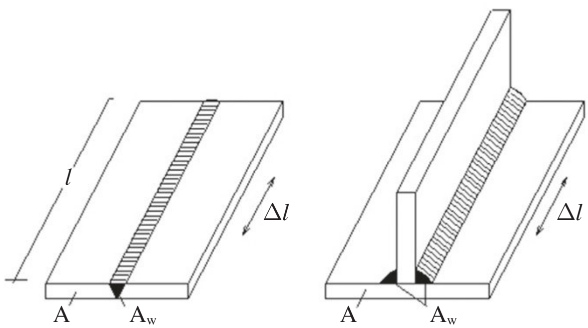


Fig. 5.20. Shrinkage in longitudinal direction

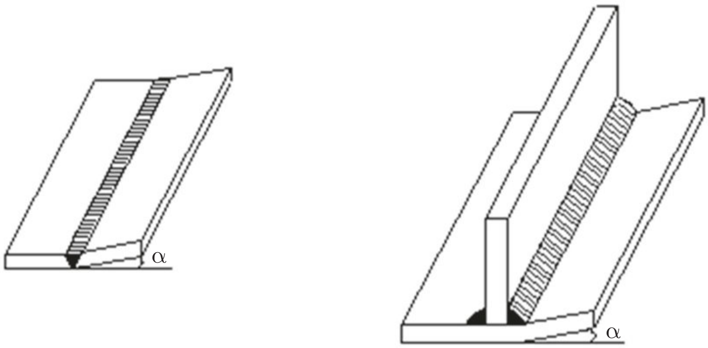


Fig. 5.21. Angular distortion for butt and fillet welds

Angular distortion takes place both in butt and in fillet welds due to un-symmetric temperature development, Figure 5.21. In both cases the plates converge towards the side that is cooling last. In butt welds this is the side where the width of the weld is larger. The distortion angle depends on the width difference between top and bottom side of the weld and consequently on the plate thickness and type of weld. Thicker, one-sided welds lead obviously to larger distortion. A symmetrical arrangement, like an X-, or U-weld or a V-weld with back welded root leads to much smaller distortion than a V-weld welded from one side. The angle of angular distortion for butt welds may be up to $\alpha = 15^0$.

Angular distortion takes place also in fillet welded T-joints, where the plates converge towards the T-stem. The angle of angular distortion increases with the ratio between weld and plate thickness and the type of welding and may be up to 7^0. In order to minimize the effects of angular distortion the plates must be provided before welding with an initial angle opposite to the expected distortion.

During cooling, shrinkage of the weld is restrained so that the weld is subjected to a tension force while for equilibrium reasons the member is subjected to an equal compression force. If the weld is eccentric to the cross-section centroid G, so is this compression force and the result is a bow deformation of the member towards the centroid G, see Figure 5.22. The amplitude of this deflection depends on the eccentricity, the thickness and type of weld, as well as the stiffness and length of the member. A countermeasure against bow deformation is to pre-camber the member before welding in opposite direction to the expected deformation.

Fig. 5.22. Bow welding deflection

5.5 Design of joints

5.5.1 Long lap joints

Generally, the distribution of stresses in lap joints is not uniform along the bolts or welds in the direction parallel to the applied force, Figure 5.23, but at the ends the forces are larger and decrease towards the middle. Theoretically, if the bolts were infinite rigid and the plates deformable the entire force would be resisted by the edge bolts only. On the other side if the bolts were deformable and the plates rigid, the forces would be equal. In reality, the edge bolts receive the largest forces at the initial loading steps. By further increase of the applied forces, the edge bolts subject to plastic deformations so there is a shedding of forces from the most stressed, edge, bolts to the least stressed, inner, bolts leading to almost equal forces to all bolts at the ultimate limit state. However, this redistribution has a limit and cannot fully compensate the initial non-uniformity for long joints. In order to keep the design equations unchanged, a reduction factor β_{Lf} is applied to the shear resistance $F_{v,Rd}$ of all bolts when the joint is long such as $L_j > 15d$ which is determined from [5.2]:

$$0.75 \leqslant \beta_{Lf} = 1 - \frac{L - 15d}{200d} \leqslant 1.0 \qquad (5.26)$$

where

L is the length of the joint and
d the diameter of the bolts.

This reduction does not apply to the bearing resistance due to the fact that bearing is a ductile failure mode allowing plastic redistribution.

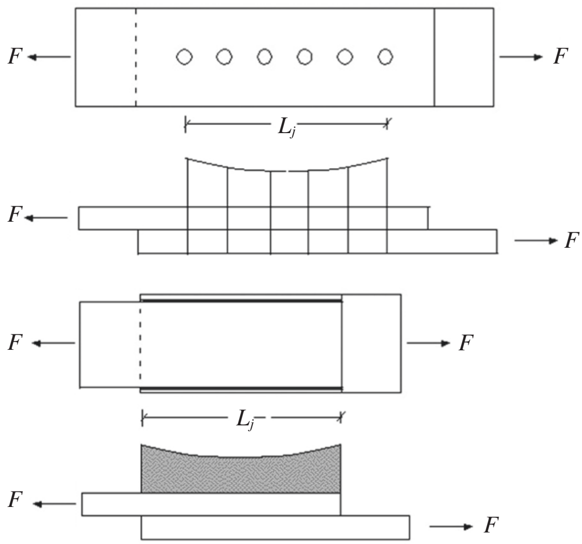


Fig. 5.23. Distribution of forces in bolts and forces per unit length in welds in long joints

Similar conditions hold for long welded lap joints with $L_j > 150a$, where a reduction factor $\beta_{Lw.1}$ is applied to the design shear strength of the weld $f_{vw,d}$:

$$\beta_{Lw,1} = 1.2 - 0.2L_j/(150a) \leqslant 1 \tag{5.27}$$

where a is the thickness of the fillet weld.

For plated members with transverse stiffeners welded by fillet welds which are longer than 1.7 m, the reduction factor is written as:

$$0.60 \leqslant \beta_{Lw,2} = 1.1 - \frac{L_w}{17} \leqslant 1.0 \tag{5.28}$$

where L_w is the length of the weld in meters.

5.5.2 Splices of members

Members are frequently spliced by splice plates in the flanges and the web, which are bolted or welded to the flanges and the web of the initial cross-section, Figure 5.24. Beam splices may be full or partial strength, i.e. the splice may be stronger or not to the initial cross-section. In full strength splices the strength of the splice plates and the connecting media is at least equal to the strength of the corresponding part of the section. This means that the product of their area with the yield strength is higher than the corresponding one of the initial cross-section. Such a design leads often to too thick and long splice plates so that in practice partial strength splices may be used, able to transfer the applied forces and moments at the position of the splice. In addition, the splice should have some minimum dimensions, e.g. the splice plates should have at least half of the thickness of the connected parts.

For splices of compression columns, part of the forces may be considered to be transferred through contact so that only flange splice plates may be used, while web splice plates may be avoided [5.28].

In beam splices, the splice plates transfer forces and moments that are resisted by the part of the cross-section they join. Accordingly, the plates that join the flanges have to transfer the partial forces that are resisted by the flange of the cross-section, while the plates joining the web by the partial forces and moments resisted by the web. The partial forces may be determined considering elastic behavior of the cross-section. In the following the partial forces and moments resisted by the flanges and the web of an I-cross-section subjected to axial forces, shear forces and bending moments will be determined, Figure 5.24:

- Design axial force N_{Ed}

The partial forces transferred by the flanges and the web are as follows:

$$\text{Flanges:} \qquad N_{f,N} = \frac{A_f}{A} \cdot N_{Ed} \tag{5.29}$$

$$\text{Web:} \qquad N_{w,N} = \frac{A_w}{A} \cdot N_{Ed} \tag{5.30}$$

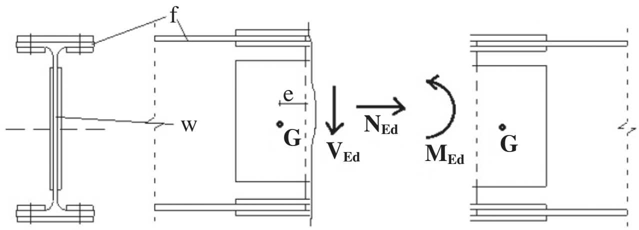


Fig. 5.24. Splice of *I*-sections. Forces and moments shown act at the left part of the splice

where:

A_f is the cross-section area of one flange
A_w is the cross-section area of the web
A is the area of the entire cross-section

If N_{Ed} is a compression force, part of it is transferred through contact.

- Design shear force V_{Ed}

The entire shear force is transferred by the web so it is:

$$\text{Web:} \qquad V_w = V_{Ed} \tag{5.31}$$

Since this force applies at the axis of the splice, it has an eccentricity e in respect to the centroid G of the half splice plate and the connecting bolts or welds, Figure 5.24. Accordingly, it creates an additional moment $V_{Ed} \cdot e$ that has to be resisted by the connection.

- Design bending moment M_{Ed}

Considering an elastic stress distribution due to the applied moment, Figure 5.25, it may be seen that the flanges are transferring axial forces and the web an axial force and a bending moment. The partial forces are determined as follows:

$$\text{Flanges:} \qquad N_{f,M} = \frac{S_f}{I} \cdot M_{Ed} \tag{5.32}$$

$$\text{Web:} \qquad N_{w,M} = \frac{S_w}{I} \cdot M_{Ed} \tag{5.33}$$

$$M_{w,M} = \frac{I_w}{I} \cdot M_{Ed} \tag{5.34}$$

where:

S_f is the first moment of area of the relevant flange in respect to the cross-section centroid
S_w is the first moment of area of the web in respect to the cross-section centroid
I_w is the second moment of area of the web in respect to its own centroid
I is the second moment of area of the entire cross-section.

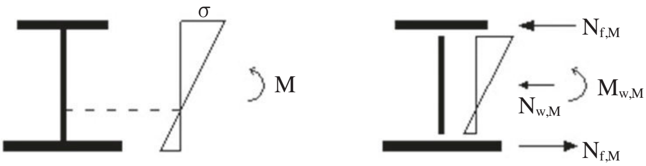


Fig. 5.25. Elastic stress distribution and partial forces and moments due to M_{Ed}

5.5.3 Groups of fasteners

5.5.3.1 Groups of fasteners under in-plane moment or eccentric force

This case appears for example in web splice plates subjected to partial moments M_w, Figure 5.25, or moments arising from an eccentric force, Figure 5.24. The forces on the fasteners which resist the applied moment may be determined by an elastic analysis method, where each force is perpendicular to the line connecting it with the centroid of the fasteners and proportional to the distance r_i, Figure 5.26a. This method may be applied always, but it has to be used for slip resistant category B or C connections, or when the shear capacity is critical.

Alternatively, a plastic analysis method may be applied, where any possible force distribution that satisfies equilibrium, such as indicatively shown in Figure 5.26b, may be assumed. The connection design is as follows.

- Elastic design (slip resistant connections or critical shear resistance)

Fastener forces:

$$F_i = \frac{r_i}{\Sigma r_i^2} \cdot M_{Ed} \qquad (i = 1 \text{ to } n,\ \ n = \text{number of fasteners}) \tag{5.35}$$

where:

r_i = distance of fastener from the rotation center (fasteners centroid G)

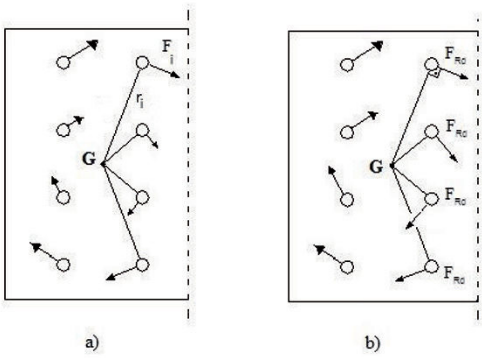


Fig. 5.26. Fastener forces according to a) elastic and b) plastic analysis methods

Design of fasteners:

$$\max F_i \leqslant F_{v,Rd} \quad \text{or} \quad F_{s,Rd} \tag{5.36}$$

- Plastic design (critical bearing capacity)

A possible force distribution that satisfies equilibrium is shown in Figure 5.26b. This distribution assumes forces equal to the bearing resistance and direction of forces perpendicular to the line connecting them to the rotation center.

Design:

$$M_{Ed} \leqslant M_{Rd} = \Sigma F_{b,Rd} \cdot r_i \tag{5.37}$$

It is mentioned that for application of a pure moment, the sum of the fastener force components in both horizontal and vertical axes is zero. However, when the moment results from an eccentric force the fastener forces must equilibrate both the applied force and the applied moment. The force resultants are determined as the sum of the two components as illustrated in Figure 5.27.

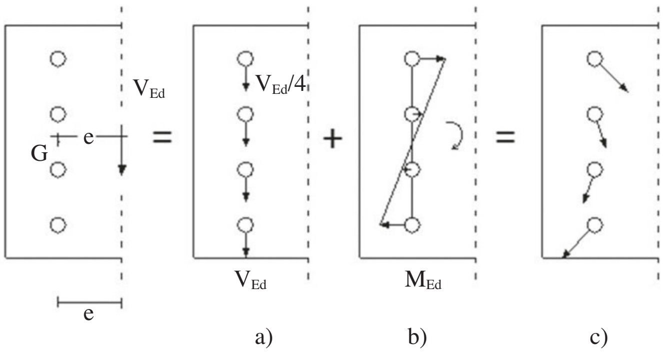


Fig. 5.27. Fastener forces a) for concentric force, b) for moment, c) force resultants

5.5.3.2 Block tearing

For more fasteners in a group the possibility of block tearing must be examined as a possible failure mode, besides the shear and bearing capacity of individual fasteners [5.29]. Examples of block tearing, or block shear according to US Codes [5.30], [5.31], are illustrated in Figure 5.28. It may be seen that a full material piece of the connection parts is separated along the bolt lines due to tensile rupture or shear yielding. The design resistance may be determined as follows [5.2].

- Concentric loading on a symmetric bolt group (Figure 5.28c)

$$V_{\text{eff},1,Rd} = \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}} \tag{5.38}$$

- Eccentric loading (Figures 5.28a, b)

$$V_{\text{eff},2,Rd} = 0.5 \cdot \frac{f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{f_y \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}} \tag{5.39}$$

where:

A_{nt} is the net area subjected to tension and
A_{nv} is the net area subjected to shear.

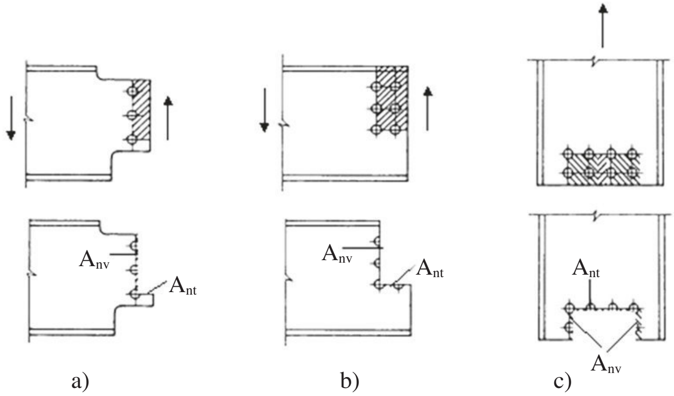


Fig. 5.28. Examples of block tearing

5.5.4 *T*-stubs

5.5.4.1 T-stubs under tension

A basic component for more elaborated joints is the T-stub and the L-stub joint, Figure 5.29. T-stub joints have bolts on two sides and are appropriate for connections with open I-sections, while L-stub joints have bolts on one side and are appropriate for connections with hollow sections, angles or even for I-sections as in column bases. In the following the resistance of the T-stub will be presented, the one of the L-stub may be determined analogously.

The T-stub resistance to tension force may be determined as the minimum from the three possible mechanisms as illustrated in Figure 5.30 and summarized in Table 5.17. Mechanism 1 corresponds to yielding of the plates by formation of four plastic hinges, mechanism 2 to bolt failure with yielding of the plates by formation of two plastic hinges and development of prying forces and mechanism 3 to bolt tensile fracture. The resistances are determined by equating the internal with the external work. For mechanism 1 the internal work is due to the rotation ϕ of the four plastic hinges, $4 \cdot M_{pl} \cdot \phi$, and the external work due to the displacement δ of the applied force, $F_T \cdot \delta$, Figure 5.30. Introducing $\delta = \phi \cdot m$ in the second relation, where m is the distance between plastic hinges, and equating the two works gives $4 \cdot M_{pl} \cdot \phi = F_T \cdot \phi \cdot m$ and consequently the T-stub resistance for mechanism 1 as indicated in the 1st row of Table 5.17. The resistances for the other two mechanisms may be derived analogously.

In a beam-to-column bolted end-plate connection two T-stubs must be examined: a T-stub for the end-plate and a T-stub for the column flange, Figure 5.29. For this connection the resistance of each T-stub must be calculated independently and the final resistance be determined as the minimum value between the two T-stubs. For

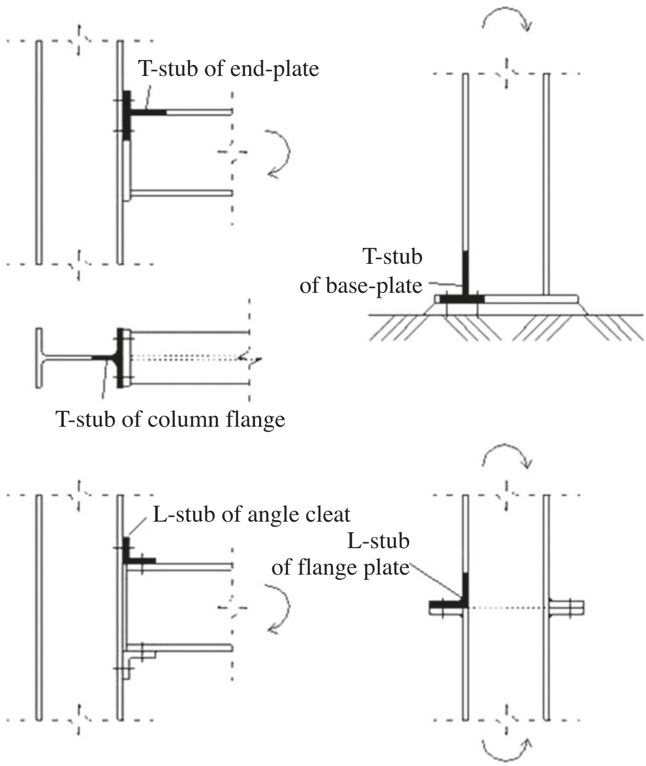


Fig. 5.29. T- and L-stub joints as components of more elaborated joints

a plastic hinge that develops over the length l in a plate of thickness t, the plastic moment is equal to $M_{pl} = b \cdot l^2/4$. Table 5.17 introduces an effective length l_{eff} in order to consider the various joint configurations covered by Table 5.17.

For a stiffened column flange reference is made to the provisions of EN 1993-1-8.

Table 5.17. T-stub design resistance

Mechanism 1 (weak flanges)	$F_{T,1,Rd} = \dfrac{4M_{p\ell,1,Rd}}{m}$
Mechanism 2	$F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$
Mechanism 3 (weak bolts)	$F_{T,3,Rd} = \Sigma F_{t,Rd}$
Final T-stub resistance	$F_{T,Rd} = \min\{F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}\}$

Notation (Symbols in Figure 5.30)
$M_{pl,i,Rd} = \dfrac{\Sigma l_{\text{eff},i} \cdot t^2}{4} \cdot f_y/\gamma_{M0}$, $(i = 1,2)$ plastic moment for the plate
f_y yield strength of plate
$\sum l_{\text{eff}i}$ effective length for mechanisms 1 or 2, Tables 5.18, 5.19
$\Sigma F_{t,Rd}$ design tension resistance of all bolts in the T-stub

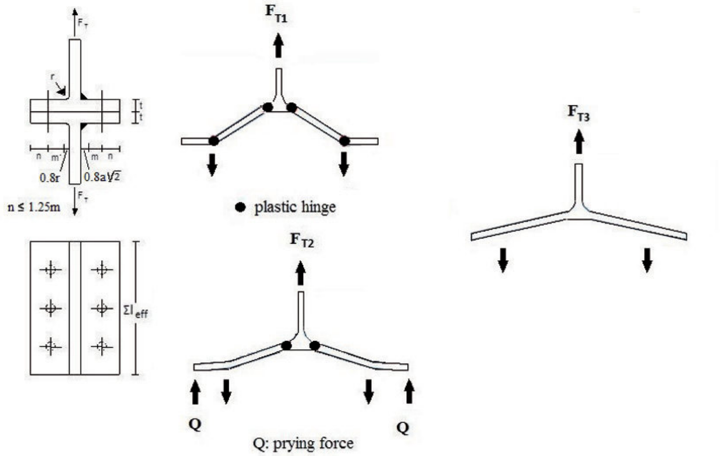


Fig. 5.30. Failure mechanisms for T-stub joints in tension

Table 5.18. Effective lengths for an end-plate [5.2]

Location of bolt-row	*Bolt-row considered individually*		*Bolt-row considered as part of bolt-rows*	
	Circular patterns $l_{\text{eff},cp}$	Non-circular patterns $l_{\text{eff},nc}$	Circular patterns $l_{\text{eff},cp}$	Non-circular patterns $l_{\text{eff},nc}$
A: External bolt-row	$\min\{2\pi m_x;\ 2\pi m_x + w;\ 2\pi m_x + 2e\}$	$\min\{4m_x + 1.25e_x;\ e + 2m_x + 0.625e_x;\ 0.5b_p;\ 0.5w + 2m_x + 0.625e_x\}$	–	–
B: 1st internal bolt row	$2\pi m$	αm	$\pi m + p$	$0.5p + \alpha m - (2m + 0.625e)$
C: Other inner bolt row	$2\pi m$	$4m + 1.25e$	$2p$	p
D: Other end bolt-row	$2\pi m$	$4m + 1.25e$	$\pi m + p$	$2m + 0.625e + 0.5p$
Mechanism 1	$l_{\text{eff},1} = \min\{l_{\text{eff},nc};\ l_{\text{eff},cp}\}$		$\Sigma l_{\text{eff},1} = \min\{\Sigma l_{\text{eff},nc};\ \Sigma l_{\text{eff},cp}\}$	
Mechanism 2	$l_{\text{eff},2} = l_{\text{eff},nc}$		$\Sigma l_{\text{eff},2} = \Sigma l_{\text{eff},nc}$	

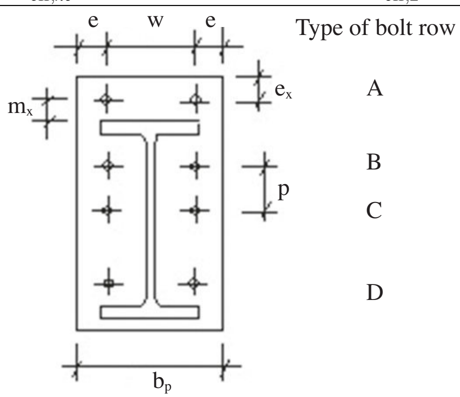
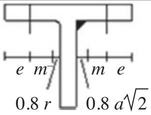


Table 5.19. Effective lengths for an unstiffened column flange [5.2]

Location of bolt-row	*Bolt-row considered individually*		*Bolt-row considered as part of bolt-rows*	
	Circular patterns $l_{\text{eff},cp}$	Non-circular patterns $l_{\text{eff},nc}$	Circular patterns $l_{\text{eff},cp}$	Non-circular patterns $l_{\text{eff},nc}$
Inner bolt-row	$2\pi m$	$4m + 1.25e$	$2p$	p
end bolt row	$\min\{2\pi m;\ \pi m + 2e_1\}$	$\min\{4m + 1.25e;\ 2m + 0.625e + e_1\}$	$\min\{\pi m + p;\ 2e_1 + p\}$	$\min\{2m + 0.625e + 0.5p;\ e_1 + 0.5p\}$
Mechanism 1	$l_{\text{eff},1} = \min\{l_{\text{eff},nc};\ l_{\text{eff},cp}\}$		$\Sigma l_{\text{eff},1} = \min\{\Sigma l_{\text{eff},nc};\ \Sigma l_{\text{eff},cp}\}$	
Mechanism 2	$l_{\text{eff},2} = l_{\text{eff},nc}$		$\Sigma l_{\text{eff},2} = \Sigma l_{\text{eff},nc}$	



5.5.4.2 *T-stubs under compression*

T-stubs appear also in column bases, Figure 5.29. For large compression forces and small moments, the T-stubs may be also under compression. In such cases the flange compression forces disperse under the base plate and are resisted by the concrete or the grout material. The dispersion width, c, is determined from the condition that a plastic moment develops in the base plate. The applied moment per unit length at the interface to the column flange is $M_{Ed} = f_{jd} \cdot c^2/2$, while the plastic hinge moment is $M_{pl,Rd} = t^2/4 \cdot f_y/\gamma_{M0}$. Equating the applied and the resisting moments gives $c = t\sqrt{f_y/(2 \cdot f_{jd} \cdot \gamma_{M0})}$. EN 1993-1-8 proposes a similar formula, where the factor 2 is substituted by the factor 3 for additional safety, so that the dispersion width writes as in equation (5.40). Obviously, c is limited by the actual base plate dimensions.

$$c = t\sqrt{f_y/(3 \cdot f_{jd} \cdot \gamma_{M0})} \tag{5.40}$$

where:

- t is the thickness of the base plate, Figure 5.31
- f_y is the yield strength of the base plate
- f_{jd} is the design bearing strength of the grout.

The design compression strength of the T-stub is then determined from:

$$F_{c,Rd} = f_{jd} \cdot b_{\text{eff}} \cdot l_{\text{eff}} \tag{5.41}$$

5.5.5 Beam-to-column joints

Beam-to-column joints are very important elements for a steel structure in respect to strength and stiffness. The joint configuration depends on many parameters such as

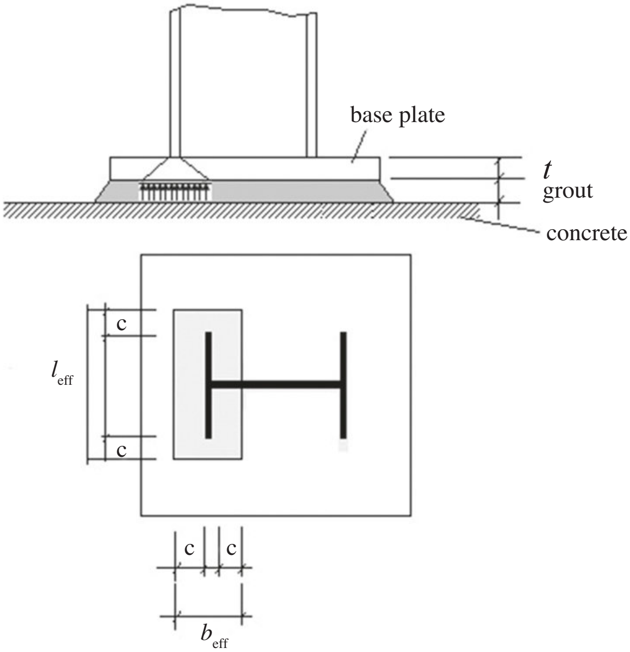


Fig. 5.31. Effective area under a T-stub in compression

the connection type, welded or bolted, the column shape, I- or hollow sections, the axis of the I-column where the beam is connected, strong or weak axis, the angle of inclination between the beam and the column or other. Some typical configurations are illustrated in Figure 5.32.

Conventionally, beam-to-column joints are considered in analysis and design either as pinned or as rigid. However, in real practice they behave rather semi-rigid, i.e. between the two extreme cases allowing relative rotation between the connected members and developing moments. In the following typical welded and bolted beam-to-column joints will be presented, where both beam and columns are of I-section and the beam is connected to the strong axis of the column cross-section. The joint resistance is determined according to the component method as introduced in EN 1993-1-8. This method studies the stiffness and strength of individual components of the joint and assembles them together to determine the properties of the joint.

5.5.5.1 Welded beam-to-column joints

Figure 5.33 shows a joint between I-shaped beams and columns, where the beam flanges and the web are welded to the column flange. The individual components for a joint subjected to a bending moment are the column web in tension, compression and shear and the beam flange and web in compression.

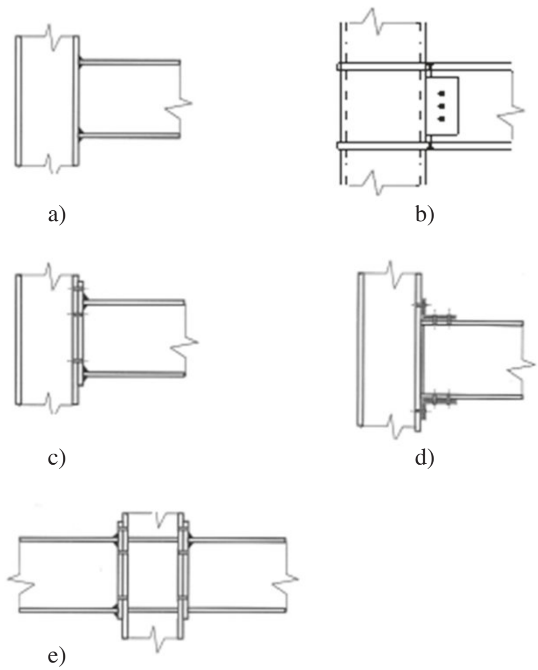


Fig. 5.32. Typical beam-to-column joints. a) welded edge joint, b) welded flanges with shear tabs for hollow section columns, c) bolted end-plate, d) angle cleats, e) internal joint with stiffeners and bolted end-plate

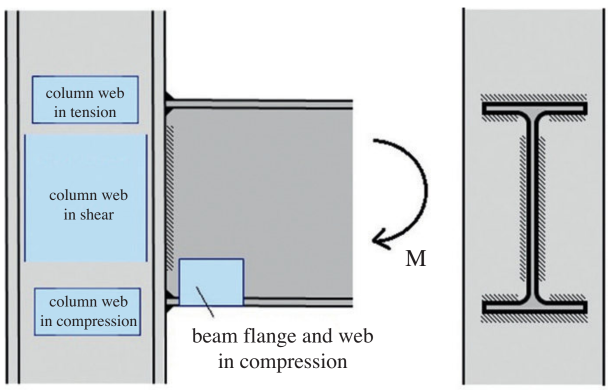


Fig. 5.33. Components of a welded beam-to-column joint

- Column web in compression

The load transfer from the beam flange to the column web is shown in Figure 5.34. The contact area includes the beam flange and the welds. Subsequently the stresses are dispersed, assuming an inclination angle 2.5 : 1 until they reach the net column

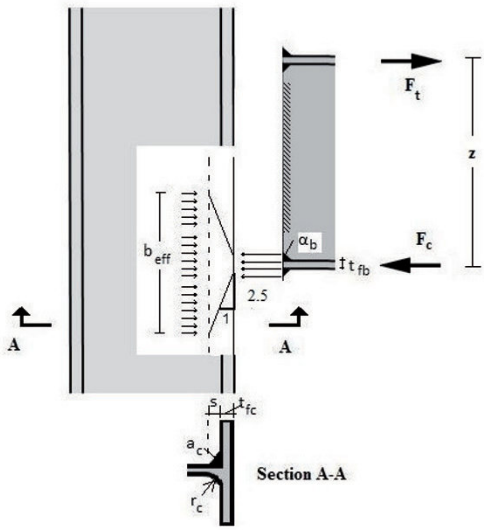


Fig. 5.34. Stress distribution in the tension zone of the column web

web thickness. The possible failure modes are yielding or buckling of the column web.

- Column web in tension

The load transfer is similar to Figure 5.34, with opposite direction of stresses. The failure mode is yielding of the column web.

- Column web in shear

The analytical model considers the column section as subjected to a concentrated shear force resulting from the compression or tension force transferred from the beam flanges.

Table 5.20 presents the resistances of the three components as proposed by EN 1993-1-8. For more detailed information, reference is made to the provisions of the Code.

The moment resistance of the joint is calculated from following expression, Figure 5.34:

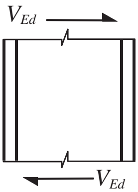
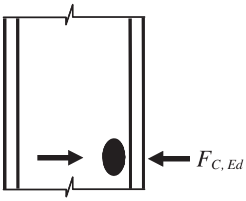
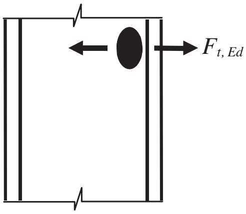
$$M_{j,Rd} = F_{Rd} \cdot z \tag{5.42}$$

where

$F_{Rd} = \min\{V_{wp,Rd}, F_{c,wc,Rd}, F_{t,wc,Rd}\}$ = minimum resistance of the 3 zones
z = lever arm (centroid distance between beam flanges)

For joint configurations with stiffeners some design checks need not be made, or the resistances increased. If horizontal stiffeners are added in the column web in extension of the beam flanges, the relevant compression or tension checks need not

Table 5.20. Design resistances of basic components for unstiffened welded beam-to-column joints

Component		*Resistance*
Column web in shear		$V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\,\gamma_{M0}}$ A_{vc} = shear area of column. Condition: no shear buckling
Column web in compression		$F_{c,wc,Rd} = \frac{\omega k_{wc} b_{\mathrm{eff},c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$, but $F_{c,wc,Rd} \leqslant \frac{\omega k_{wc} \rho\, b_{\mathrm{eff},c,wc} t_{wc} f_{y,wc}}{\gamma_{M1}}$ where $b_{\mathrm{eff},c,wc} = t_{fb} + 2\sqrt{2}a_b + 5(t_{fc} + s)$
Column web in tension		$F_{t,wc,Rd} = \frac{\omega b_{\mathrm{eff},t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$ *where* $b_{\mathrm{eff},t,wc} = t_{fb} + 2\sqrt{2}a_b + 5(t_{fc} + s)$
Beam flange and web in compression		$F_{c,fb,Rd} = \frac{M_{b,Rd}}{h - t_{fb}}$

Notation

Geometric properties according to Figure 5.34
– For column with rolled section: $s = r_c$
– For column with welded section: $s = \sqrt{2}\alpha_c$
ω reduction factor to consider an interaction with shear in column from [5.2]
ρ reduction factor to consider compression buckling in the column web from [5.2]
k_{wc} reduction factor to consider interaction with compression stresses along the column due to axial force and bending.
$M_{b,Rd}$ Design bending resistance of beam, possibly reduced due to shear
h height of beam

be made since the stiffeners transfer the concentrated forces. In addition, the shear resistance is increased due to formation of plastic hinges in the column flanges. If a diagonal stiffener is provided in the column that transfers the shear force, no shear check is necessary. If a supplementary web plate is used to reinforce the web, its area may be added to the shear area of the column.

5.5.5.2 Bolted end-plate beam-to-column joints

A typical configuration of a bolted joint between I-shaped beams and columns, where the connection is provided through an end-plate that is welded to the beam end and bolted to the column flange is illustrated in Figure 5.35. The individual components for such a joint are similar in respect to the column, i.e. column web in tension, compression and shear, and the compression zone of the beam. However, the tension zone behaves like two T-stubs, one referring to the column flange and one referring to the end-plate. The design resistances may be accordingly determined following the rules presented before. The determination of the T-stub resistances is laborious due to the fact that two distinct T-stubs with different properties concerning the geometric dimensions of plates and the positions of bolts in the end-plate and the column flange and different positions of bolt rows, external, internal close to the tension flange, other internal rows etc. must be considered. Hand calculations are almost prohibited due to possible calculation errors and the designer must rely more or less on software programs.

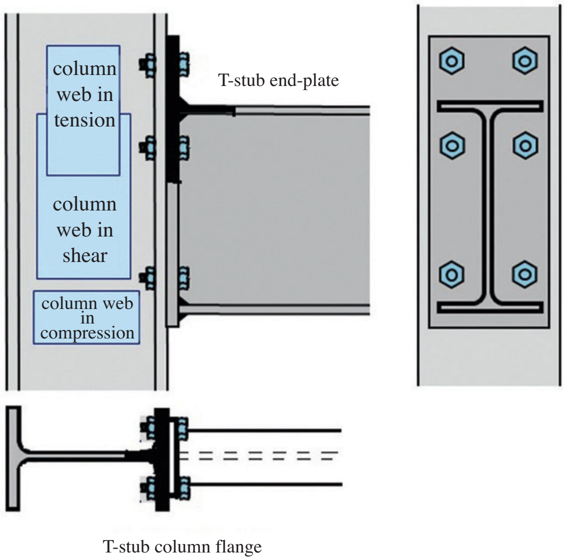


Fig. 5.35. Components of a bolted end-plate beam-to-column joint

The calculation delivers the design resistance for the individual bolt rows, the compression zone, the tension zone and the shear zone. The moment resistance is determined from equilibrium of force and moments. For the conditions shown in Figure 5.36, the force equilibrium between the compression force and the bolt forces requires $F_c = F_{t1} + F_{t2}$. Accordingly, the moment resistance may be determined as following:

$$M_{j,Rd} = F_{t1,Rd} \cdot h_1 + F_{t2,Rd} \cdot h_2 \tag{5.43}$$

but

$$F_{t1,Rd} + F_{t1,Rd} \leqslant \min\{V_{wp,Rd}; F_{c,wc,Rd}; F_{t,wc,Rd}\}$$

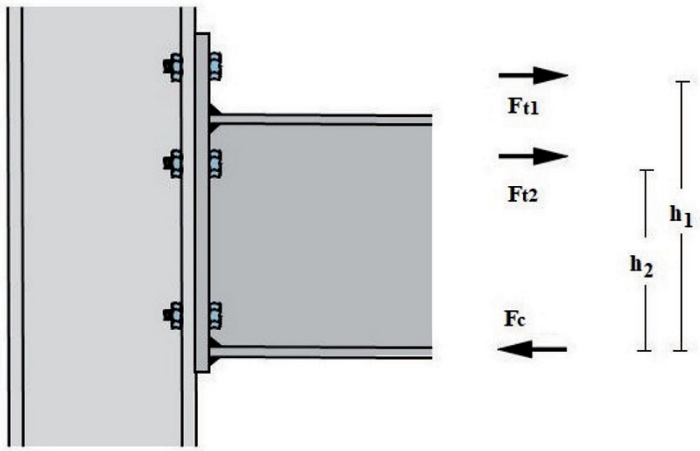


Fig. 5.36. Forces and lever arms in a bolted beam-to-column joint

5.5.5.3 Joint modelling and stiffness

Conventionally, joints are modelled either as hinges or as rigid. However, the real behavior is different and joints behave as semi-rigid. In order to capture the realistic joint behavior, a model should be used in which the various zones are introduced separately and represented by appropriate springs. Figure 5.37 shows such representation for welded and bolted joints. For linear analysis each spring is introduced by its stiffness K, while the resulting forces of the springs must be checked against the corresponding design resistances. Table 5.21 gives values of the spring stiffness for various zones and individual elements. Further information for semi-rigid joints is given in [5.32] to [5.34].

Alternatively, a single rotation spring of equivalent stiffness may be introduced between the column and the beam. For welded joints, Figure 5.37, the equivalent elastic stiffness is determined from:

$$S_{j,ini} = \frac{z^2}{\frac{2}{K_{cws}}} + \frac{1}{K_{cwc}} + \frac{1}{K_{cwt}} \tag{5.44}$$

For bolted joints a similar expression is used in which an equivalent stiffness for the tension zone K_{eq} is introduced that takes into account all flexibilities from the T-stubs. Accordingly, it is:

$$S_{j,ini} = \frac{z^2}{\frac{2}{K_{cws}}} + \frac{1}{K_{cwc}} + \frac{1}{K_{eq}} \tag{5.45}$$

The equivalent stiffness may be determined from, Figure 5.37:

$$K_{eq} = \sum \left(\frac{1}{K_{cwt}} + \frac{1}{K_{cfb}} + \frac{1}{K_{bt}} + \frac{1}{K_{epb}} \right)_j \tag{5.46}$$

where the summation refers to the bolt rows under tension.

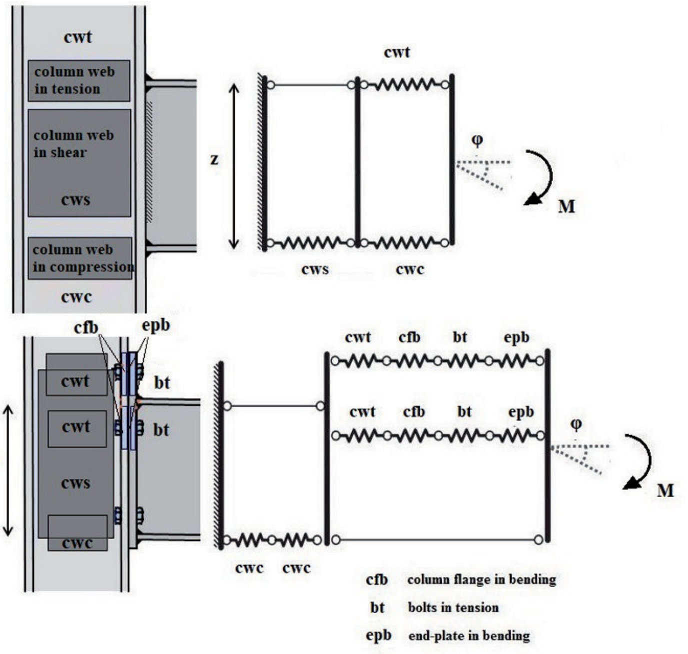


Fig. 5.37. Elaborate models for representation of semi-rigid joints

Table 5.21. Spring stiffness of joint components

Component	*Stiffness K*	*Notation*
Column web in shear	$K_{cws} = E \cdot \dfrac{0.38A_v}{\beta \cdot z}$	$\beta = 1$ for edge joints and $0 \leqslant \beta \leqslant 2$ for internal joints, z lever arm
Column web in tension	$K_{twc} = E \cdot \dfrac{0.7 \cdot b_{\text{eff},t,wc} \cdot t_{wc}}{d_c}$	d_c clear depth of column web
Column web in compression	$K_{cwc} = E \cdot \dfrac{0.7 \cdot b_{\text{eff},c,wc} \cdot t_{wc}}{d_c}$	
Column flange in bending (one bolt-row in tension)	$K_{cfb} = E \cdot \dfrac{0.9 l_{\text{eff}} \cdot t_{fc}^3}{m^3}$	m defined in Figure 5.30
End-plate in bending (one bolt-row in tension)	$K_{cfb} = E \cdot \dfrac{0.9 l_{\text{eff}} \cdot t_p^3}{m^3}$	
Bolts and anchor bolts in tension (one bolt-row, bolts preloaded or not)	$K_{bt} = E \cdot \dfrac{1.6A_s}{L_b}$	L_b bolt elongation length
Bolts in bearing for each plate of thickness t_j	$K_{bear} = 25 \cdot n_b \cdot k_b \cdot k_t \cdot d \cdot f_u$	$k_b = k_{b1} \leqslant k_{b2}$ $k_{b1} = 0.25e_1/d + 0.5 \leqslant 1.25$ $k_{b2} = 0.25p_1/d + 0.375 \leqslant 1.25$ $k_t = 1.5t_j/d_{M16} \leqslant 2.5$

5.5.5.4 Classification of joints

Joints may be classified in respect to strength and in respect to stiffness.

In respect to strength joints are full strength, partial strength or pinned as follows.

- Pinned are considered joints with a moment capacity smaller than 25% of the moment capacity of the connected members.
- Full strength are joints with a moment capacity higher than the moment capacity of the connected members.
- Partial strength are joints which are neither pinned nor full strength.

In respect to stiffness joints are considered rigid, pinned or semi-rigid as follows, Figure 5.38.

- Pinned are considered joints with an elastic stiffness

$$S_{j,ini} \leqslant \frac{0.5 \cdot E \cdot I_b}{L_b} \tag{5.47}$$

where:

E is the modulus of elasticity
I_b is the second moment of area of the connected beam
L_b is the span of the connected beam, distance between columns

- Rigid are considered joints with an elastic stiffness

$$S_{j,ini} \geqslant \frac{8 \cdot E \cdot I_b}{L_b} \quad \text{for unbraced frames} \tag{5.48}$$

or

$$S_{j,ini} \geqslant \frac{25 \cdot E \cdot I_b}{L_b} \quad \text{for braced frames} \tag{5.49}$$

Braced frames are those in which the bracing system reduces the horizontal displacements of the frame by at least 80%.

- Semi-rigid are joints which are neither pinned nor rigid.

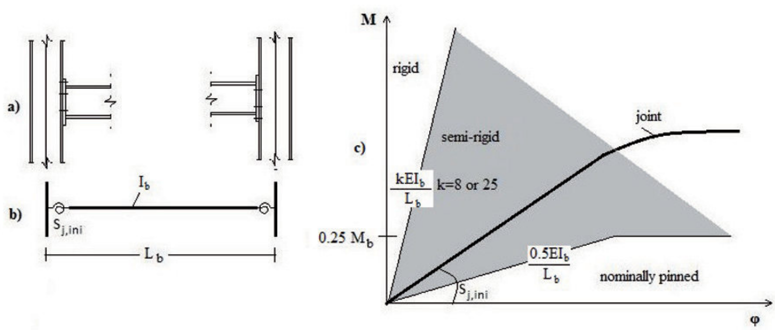


Fig. 5.38. Modeling of semi-rigid joints. a) Lay-out, b) structural model, c) definition of joint behavior

5.5.5.5 Column bases

Column bases are mostly end-plate connections, where end-plates are welded to the column and anchored in the concrete foundation. In exceptional cases columns are directly embedded in the foundation concrete. Column bases may be rigid, simple or semi-rigid. Their behavior depends on their configuration but also on the magnitude of the column axial force. Figure 5.39 shows the decomposition of the design moments and axial forces in flange axial forces that helps analysis of the joint by the component method.

If the axial forces $F_{M,Ed}$ due to bending moments are higher than the forces $F_{N,Ed}$ due to axial force, one of the flanges is in tension, the other in compression. The tension flange may be analyzed as a T-stub under tension, the compression flange as a T-stub in compression, following the rules of 5.5.4.1 and 5.5.4.2 correspondingly. Obviously, the anchors are active only in the tension flange. If the forces $F_{M,Ed}$ are smaller than $F_{N,Ed}$ all section is in compression and may be analyzed as a T-stub in compression. The cases where the design axial column force is tension may be analyzed analogously. Further information on the analytical modelling of column base connections could be found in ref [5.35] to [5.38].

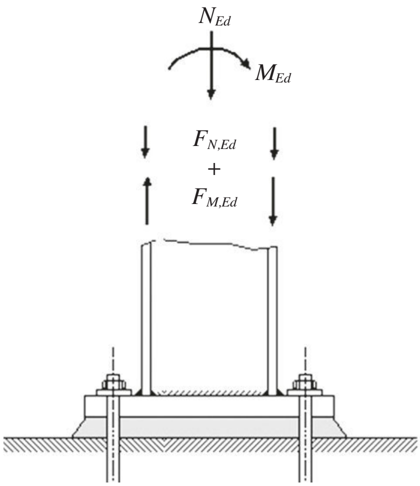


Fig. 5.39. Analysis of forces and moments in column bases

5.5.6 Hollow section welded joints

Welded joints of trusses composed of hollow cross-sections are not designed by the component method according to EN 1993-1-8. The Code offers rules for hollow sections with wall thickness between 2.5 and 25 mm that allow the determination of the axial force resistance, including in- and out-of-plane bending moments. The joints may be overlap or gap joints, without or with small eccentricities between connecting members. Members may be of CHS, RHS or SHS sections, but also combinations of hollow and I-section members are covered. The Code provides design formulae that are presented in a number of Tables. For more information, reference is made in the Code.

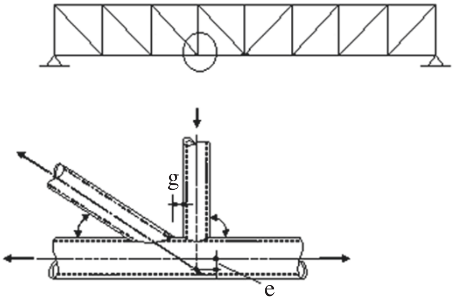


Fig. 5.40. Truss with hollow section joints

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6
Single storey buildings

Abstract. This chapter presents the main structural elements of single storey buildings, whether industrial, commercial, serving for sports activities or other use. Several structural solutions are proposed, with their advantages and disadvantages, with emphasis on the most usual ones. Reference is made to both the main and the secondary structural elements. Hot rolled, welded or cold-formed elements are described as well as cladding panels. Several types of bracing systems are also presented, such as vertical bracings, providing lateral stability to the building, wind bracings on roofs or using the panels of the skin as stability elements. Special attention is given to buildings in seismic areas, where enhanced requirements are to be met in respect to strength and ductility.

Single storey buildings are often equipped with cranes, supported by the main structure through appropriate beams. Crane supporting beams are also presented here, including the applied to them crane actions, their resistance and serviceability requirements as well as the related constructional details.

This chapter may assist designers to select, during the initial phase of the preliminary design, the main design options such as the general arrangement of the structure, structural systems, their geometry or types of cross-sections.

6.1 Typical elements of a single storey building

A typical single storey building, designed to serve industrial, commercial, sports or other activities, is shown in Fig. 6.1, where its main structural elements are also indicated. This type of building gives the possibility to create a column free space, due to the lightness of the construction and the usually small live loads, in the shortest possible time, due to the high speed of the on-site construction works. The industrial prefabrication of the steel components allows also the application of systematic quality control procedures.

The main structural system of such a building consists of portal frames arranged, in general, at equal distances. For practical and economic reasons the distance between portal frames varies between 5.0 and 8.0 m and is usually selected to be 6.0 to 6.5 m. The portal frames can resist both vertical and horizontal loads acting in their plane.

I. Vayas et al., *Design of Steel Structures to Eurocodes*, Springer Tracts in Civil Engineering, https://doi.org/10.1007/978-3-319-95474-5_6

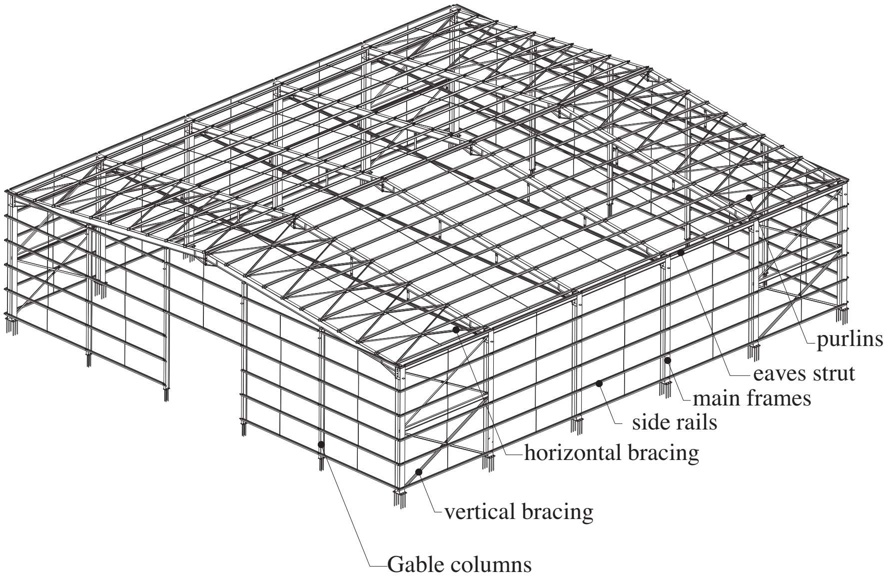


Fig. 6.1. Typical structure of a single storey steel building

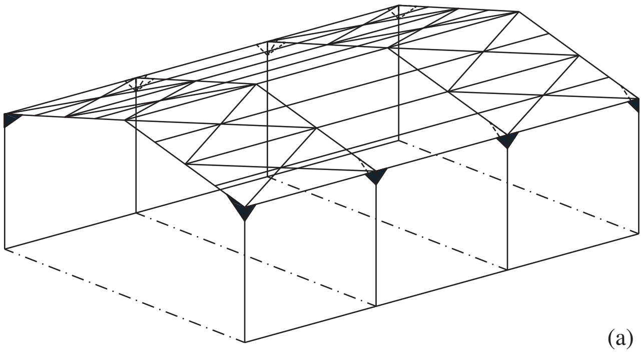
Between the portal frames are placed secondary beams, the purlins, which support the roof cladding panels and transfer to the main structure all vertical loads such as snow, wind or live loads. The distances between purlins are usually between 1.50 m and 3.00 m depending on the type of cladding panels, the distances between frames and the arrangement of the horizontal bracing system.

The lateral stability of the structure is usually ensured by means of roof and wall bracing systems. The roof, or horizontal, bracing systems are situated between frame rafters while the wall, or vertical, systems between columns. Bracing systems are usually arranged every 4 to 6 portal frames. Detailed information concerning the above bracing systems is provided in section 6.4. Element of the vertical bracings are the eave struts running along the building and joining columns' heads. These struts transfer the horizontal loads to the vertical bracings having, in addition, a very helpful role during the erection phase of the structure.

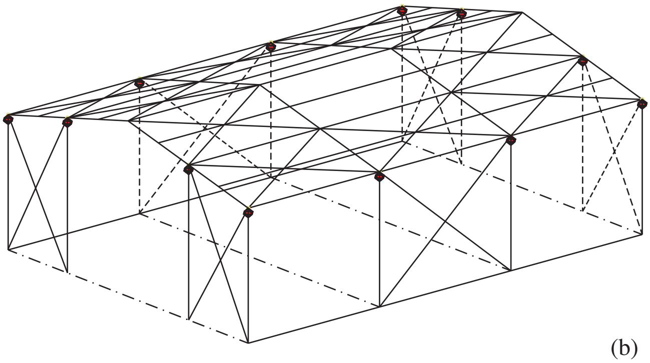
The wall cladding panels are supported by horizontal beams, the side rails, arranged between successive portal frame columns, which resist mainly the applied wind loads. The side rails are attached to the outside flange of the columns and are eccentric in respect to the main structure while the bracing elements of the vertical bracing systems are, in general, placed on the column axes being concentric to the main structure.

At the gable walls additional columns are required to support the corresponding side rails. These additional gable wall columns have usually common nodes at their top with the horizontal bracing system, in order to avoid bending effects arising from eccentricities. The gable columns could be connected to the end frames in such a way, that no vertical loads are transferred to them. Accordingly they need smaller cross-sections, since they are not subjected to compression forces, and, in addition, they don't change the structural system of the end frames. Alternatively they could be connected at their top to the end frames, through simple or fixed connections, changing their structural system.

There are alternatives to the typical structural arrangement, as described above. One is to provide rigid frame action also in the longitudinal direction and consequently to avoid completely vertical bracings (Fig. 6.2a). The second is to eliminate completely rigid frame action by use of simple beam-to-column connections for the main structural system, according to both directions, and to provide lateral stability through additional wall and roof bracing systems (Fig. 6.2b).



(a)



(b)

Fig. 6.2. Typical structure of a single storey steel building with (a) moment resisting connection along both main directions and (b) simple connections in both directions

6.2 Roofs resting on concrete columns

6.2.1 Introduction

An alternative to the all steel solution described before, is to cover a space with a steel roof on reinforced concrete columns. The steel roof is then supported by concrete beams connecting the column heads. In case of columns with substantial height intermediate beams connecting the columns could be arranged to reduce the buckling length of the columns and to facilitate the construction of the walls.

The distance between adjacent column rows, i.e. the span of the roof, may vary from 6.0 m to 50.0 m. For small spans, simple structural systems may be used, as the one shown in Fig. 6.3. This system consists of a double pitched roof with rafters and a tendon which is stressed when the roof is subjected to vertical loading. The tendon is vertically supported, at some distances, from the rafters to avoid vertical deflections due to his self-weight. For larger spans the above system is substituted by trusses.

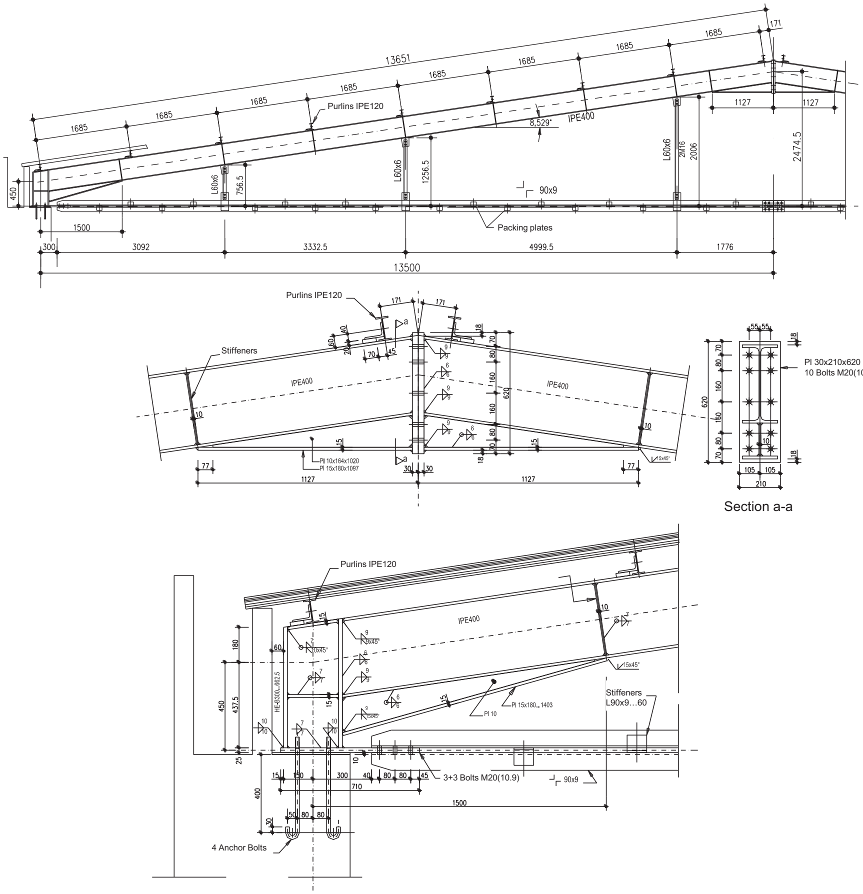
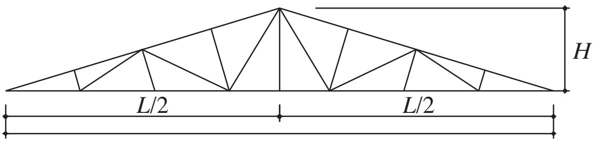


Fig. 6.3. Details of a simple structure supported by concrete elements and used for small spans

6.2.2 The geometry of the trusses

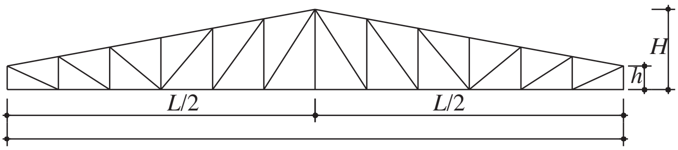
The type and geometry of trusses depends on structural, functional, constructional or cost criteria. Some typical truss forms are illustrated in Fig. 6.4. In the following some indicative design criteria are presented:

a) The slope of the upper chord varies between 2% and 20%. Larger slopes facilitate the flow of the rain waters. For small slopes efficient measures are to be taken for the waterproofing, like sufficient overlapping of the cladding panels and the use of waterproof membranes.
b) Trusses with triangular shapes (Fig. 6.4a) are used for smaller spans (indicative values 8 to 16 m) and they are related to higher upper chord slopes (up to 50%). The most compressed bars, due to vertical loading, are the ones near supports.
c) A common geometry for spans between 15 and 30 m is the N-truss, as shown in Fig. 6.4b.
d) The vertical bars (posts) are usually situated below purlins, so that vertical loads apply in nodes and bending of the upper chord bars is avoided.
e) The height of the end post is between 40 and 120 cm, as the truss span varies from 20 to 45 m (indicative values).
f) The height of the longest post, at the middle of the span, should not be greater than 4.20 to 4.50 m in order to facilitate fabrication and transport on site.



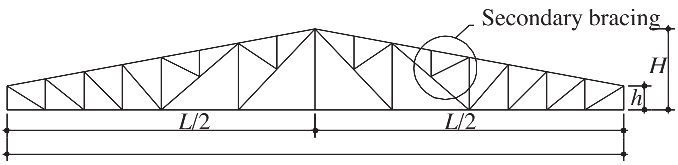
$L = (8.00\text{ m} – 16.00\text{ m})$

(a) Truss with triangular outline



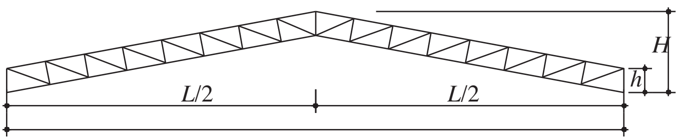
$L = (15.00\text{ m} – 30.00\text{ m})$

(b) N-Truss



$L = (30.00\text{ m} – 50.00\text{ m})$

(c) Truss with a secondary bracing



(d) Truss with parallel chords

Fig. 6.4. Indicative truss geometries

g) Bar axes should intersect at the points of the theoretical nodes. For the case of eccentricities see 6.2.4.
h) Bars should not meet each other under small angles ϕ (indicative values less than 30^0) in order to avoid the connection of diagonals at a larger distance from the nodes and consequently the necessity of large gusset plates (see also 6.2.3 and Fig. 6.6).
i) To avoid small angles as above, in trusses with large spans, a modified geometry of the secondary bars could be adopted as indicatively illustrated in Fig. 6.4c.
j) For intermediate spans, trusses with parallel chords could, as an alternative, be selected (Fig. 6.4d). The height h of the truss could be taken between 1/10 and 1/15 of the span. This type of truss is heavier than the corresponding previous types but have aesthetic and fabrication (groups of bars with the same length, typical connections) advantages.
k) The direction of the diagonals is selected such, to be subjected in tension under the most significant loading case.

6.2.3 Cross-sections of bars – Shaping of nodes

Truss bars are often formed from closely spaced built-up members that consist of two channels or two, equal or unequal legs, angles connected through packing plates (Fig. 6.5a). The angles could also be star battened by pairs of batten plates in two perpendicular directions (Fig. 6.5b). As an alternative, bars could consist of hollow sections, as discussed in the next section 6.2.4.

At the nodes the bars are connected to a gusset plate of sufficient dimensions that provide the required space for welding and bolting (Fig. 6.6). Chords are not interrupted at the nodes but they run continuously over their entire length, or the maximum length that can be transported. The center lines of the bars should meet at the theoretical point of the node, otherwise the connection is eccentric.

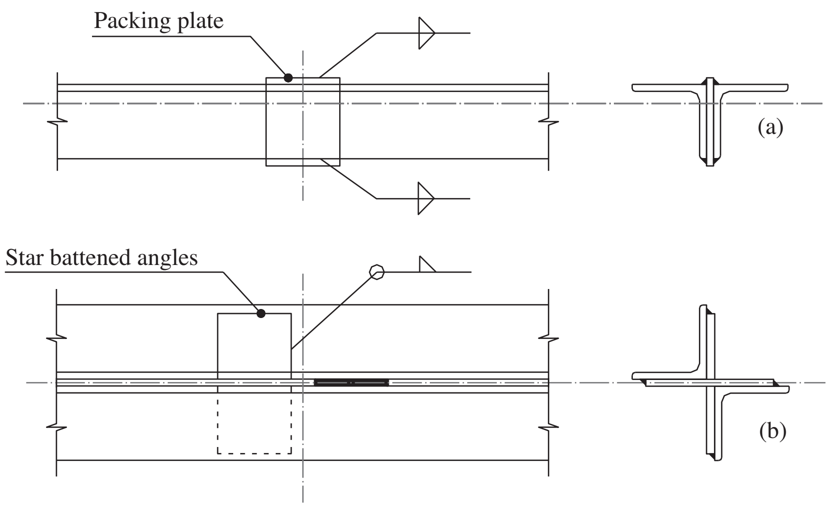


Fig. 6.5. Built-up truss bars connected through (a) packing plates (b) star battened angles

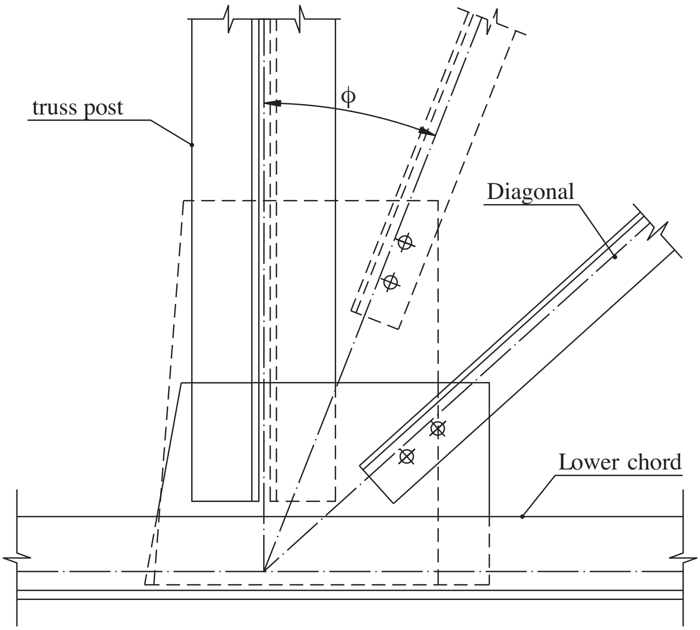


Fig. 6.6. Detail of a truss node. Diagonal under a small angle, ϕ, stops at a larger distance from the node and requires a bigger gusset plate

In case that the two chords of the built-up member are very closely connected by the packing plates, the effect of the shear flexibility could be ignored and the compressed member could be considered as a single integral member. In a different case the above effect should be taken into account. To this end, as an application rule, Eurocode 3/Part 1-1 (EN 1993-1-1) [6.1] considers the two chords as closely connected if the maximum axial spacing between interconnections is, in general, less than $15i_{min}$ or, in the case of star battened angles connected by pairs of battens, less than $70i_{min}$, where i_{min} is the minimum radius of gyration of the single cross-section.

Due to maintenance requirements of the structure, the distance between the two chords of the built-up members should not be less than a minimum dimension which could be ensured by the connecting plates. Application rules for the above minimum distance a (Fig. 6.7) are included in EN 12944/Part 3 [6.2] and presented in the section 8.5.5 of chapter 8. The distance is depended on the height h of the cross-section (Fig. 6.7) and varies from 50 mm, for $h = 100$ mm, to 300 mm, for $h = 700$ mm.

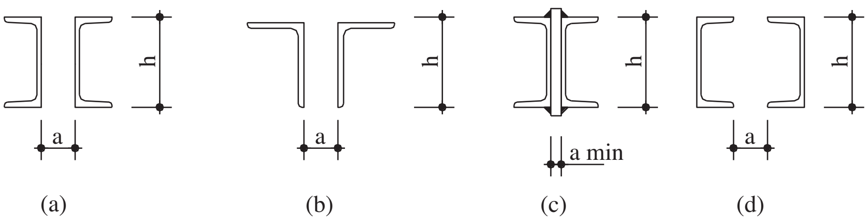


Fig. 6.7. Minimum distance between chords of built-up truss members

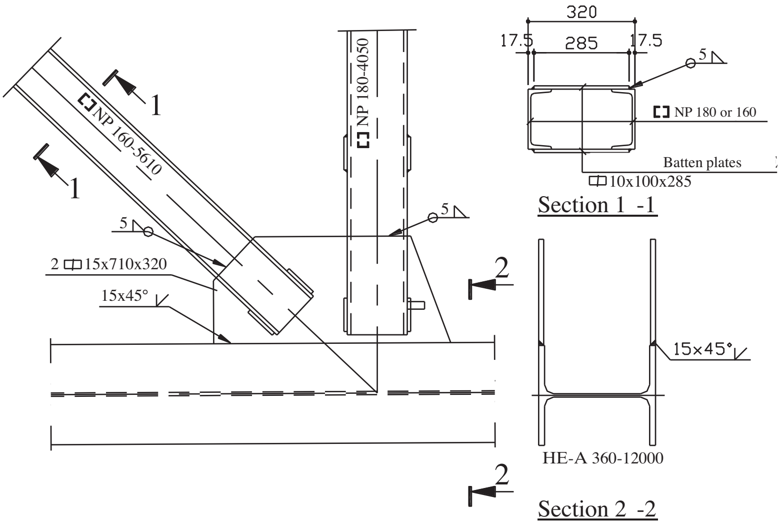


Fig. 6.8. Lower chord node detail, in truss with a large span

In case of trusses with significant spans, chords may have H-cross-sections with horizontally placed webs and brace bars as built-up members with batten plates (Fig. 6.8). This arrangement increases the out of plane buckling resistance of the lower chord, limiting the required number of lateral supports and facilitates transversal connections.

A truss should be prefabricated in smaller parts to facilitate transport to the site. For this reason the truss could be fabricated in transportable parts, within which the connections of the bars at the nodes could be executed by welding, while bolted connections should be provided for the on-site assembly of the parts to a single integral truss. An example of such a truss is shown in Fig. 6.9. However different arrangements for the connections, considering fabrication and erection criteria, might be adopted. Trusses should also be prefabricated in smaller parts, when galvanization will be applied against corrosion protection, in order to be adapted to the dimensions of galvanizing pool.

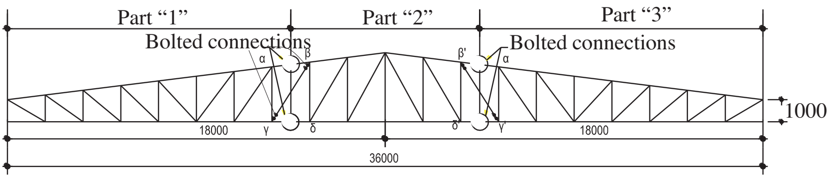


Fig. 6.9. Prefabrication of a truss in three parts

6.2.4 Trusses with hollow sections

Trusses can alternatively be composed of circular, square or rectangular hollow sections. In this case no gusset plates are used at the nodes and the bracing bars are, in general, directly welded to the chords (Fig. 6.10). For this kind of trusses the verification of the truss members should be supplemented by verification of the nodes against a local failure. This might be chord face or cross-section yielding, chord side wall failure by yielding or instability, chord shear failure, chord wall punching shear, local buckling of chord or brace member, cracking in the welds, which types of failure are not dependent on the members resistance.

The welds connecting the bracing bars to the chords should be designed with sufficient resistance to allow for non-uniform stress distributions and sufficient deformation capacity for redistribution of possible bending moments. Welds should be executed around the entire perimeter of the hollow sections using butt welds, fillet welds or combination of both, depending on the angles between members, the cross-sections thicknesses and the geometry of the node. The design resistance of the weld, per unit length of perimeter of a brace member, should not normally be less than the design resistance of the cross-section of that member per unit length of its perimeter. Characteristic indicative welding arrangement for the usual cases of nodes is shown in Fig. 6.11. Information about the joint preparation and the execution of the welding in the nodes of trusses with hollow sections is included in Annex of EN 1090-2 [6.3], related to the execution of steel structures.

Eurocode 3/Part 1.8 (EN 1993-1-8) [6.4], provides, in its chapter 7, extended and detailed application rules for the design and fabrication of uniplanar and multi-planar joints in lattice structures composed of circular, square and rectangular hollow sections. The rules are valid for steels with nominal end product yield stress not exceeding 460 MPa, for angles between chords and brace members or between adjacent brace members not less than 30^0, for brace members not thinner than 2.5 mm and for chords not thicker than 25 mm. For end products with a nominal yield stress higher than 355 MPa the static design resistance given in that chapter should be reduced by a factor of 0.90. In gap type joints (Fig. 6.10a) the clearance g is recommended

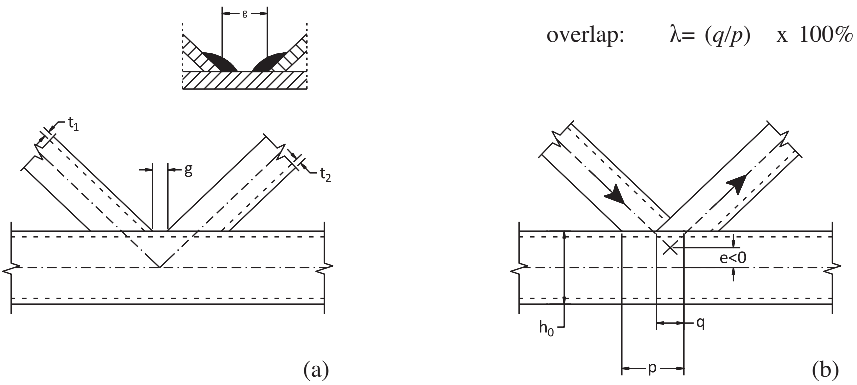


Fig. 6.10. Typical nodes in trusses with hollow sections

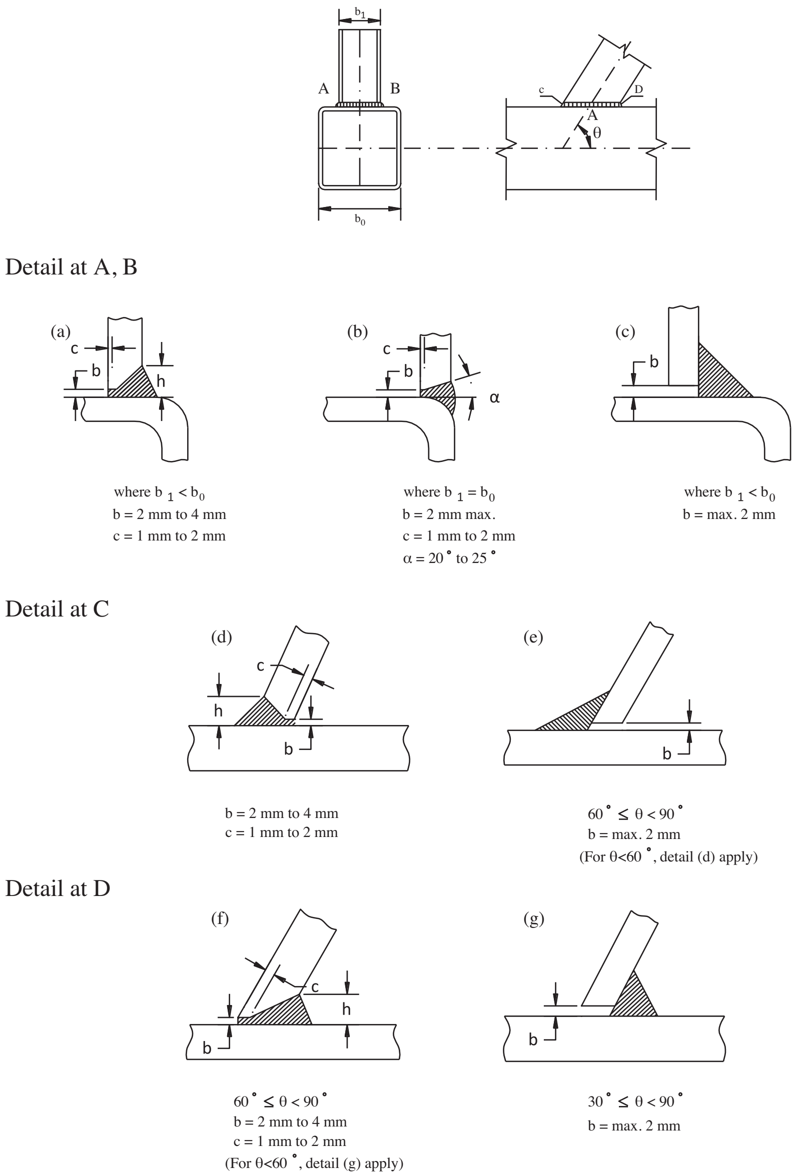


Fig. 6.11. Welded joints in lattice structures with square or rectangular hollow sections. Butt and alternative fillet welds (EN 1090-2, Annex E)

to be at least equal to $t_1 + t_2$ to ensure a satisfactory execution of the welds. When an overlapping of the bracing bars is necessary (Fig. 6.10b) the stronger bar should be overlapped by the weaker. In overlap type joints the overlapping should be sufficiently large to ensure the transfer of shear forces from one bracing member to the other. In any case the overlap should be at least 25%.

The nodes' resistances, in terms of design axial forces or moments, are given in various Tables covering all usual forms of nodes. In each Table the range of validity is also included. The above resistances are valid for members with ends prepared in such a way that their cross-sectional shape is not modified. Flattened or cropped ends are excluded. The Tables could be applied for both hot finished or cold-formed members. Rules for the application of additional reinforcing plates, when needed, are also presented.

In the detail design of the joints an eccentricity e could be introduced (Fig. 6.10b) to ensure sufficient gap or adequate overlapping. This eccentricity could be introduced in the analysis using fictitious extremely stiff members. A more extended presentation about the design of such nodes is included in edition of the Canadian Institute of Steel Construction [6.5].

According to EN 1993-1-8, the above eccentricity, for all trusses, independent to the type of the cross-sections, could be neglected in the design of tension chord members, the brace members as well as their connections, provided that it is within the range $-0.55 \leq e/h_0 \leq 0.25$ where h_0 is shown in Fig. 6.10 (depth of the chord in the plane of the truss or chord diameter in the case of a circular cross-section). The eccentricity e is considered as negative in the case shown in Fig. 6.10 and as positive when arranged on the other side of the chord's axis. However the eccentricity should be considered in the design of compression chord members.

6.2.5 Transverse connection between trusses

To ensure an overall 3D structural behavior, individual trusses should be transversely connected with structural elements as indicatively it is shown in Fig. 6.12. This type of connection is provided between all trusses except those adjacent to expansion joints. In case of smaller truss spans the transversal connection is usually placed in the middle of the trusses. For larger spans one or two additional interconnections could be provided in each half-span.

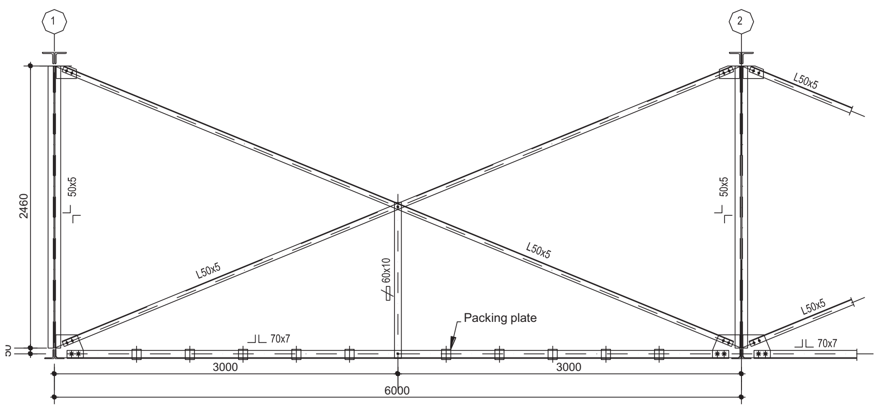


Fig. 6.12. Transverse connection between successive trusses

In case of an unforeseen overloading of one truss the above connection ensures that adjacent trusses participate in the load transfer. In addition, the above transverse connection is an important erection element ensuring, when connected, the distance between connected trusses as well their leveling. Additionally its existence excludes the in plane vibration of a single truss. The connection bars are bolted on site, through gusset plates, with the trusses. The horizontal lower bar is usually connected with a vertical bar, at its middle, to avoid deflection under its own weight (Fig. 6.12).

The transverse connection between trusses offers a lateral support to the nodes of the lower chord to which it is attached. This is very important in cases where a wind uplift which applies to the roof and the lower chord becomes a compression member with a significant buckling length in respect to the out of plane buckling. When the above buckling length should be further reduced, additional transversal bars can be arranged in intermediate locations, ending at a rigid structural element (vertical or horizontal bracing) usually at the ends of the building (Fig. 6.13).

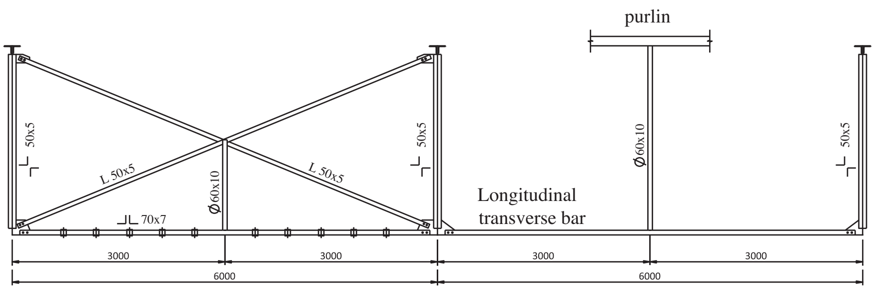


Fig. 6.13. Longitudinal bars offering lateral support to trusses nodes and ending at a rigid vertical bracing

6.2.6 Buckling length of truss bars

The buckling length for all bars of a truss may be taken equal to the system length L, where L is the distance between nodes for the in-plane buckling and the distance between lateral supports for the out-of-plane buckling.

EN 1993-1-1 [6.1] in its Annex BB, of informative character, allows, in some cases, the use of smaller buckling lengths. Accordingly, for trusses with I- or H cross-sections of the chords, the buckling length for their in-plane buckling could be taken equal to 0.90L. For bracing bars the buckling length could also be taken as 0.90L provided that the chords provide appropriate end restraint and the connections appropriate fixity (at least two bolts for bolted connections), except for angle sections where specific provisions are foreseen.

For trusses with hollow sections the buckling length for chord members could be taken equal to 0.90L, both for the in and out of plane buckling. In latticed girders with hollow sections and parallel chords, braces, for which the brace to chord width (or diameter) ratio is less than 0.60, could be calculated with a buckling length, in both buckling directions, equal to 0.75L. This is valid for bracing bars without cropping or flattening at their ends, welded according to the entire perimeter.

In any case, smaller values could be adopted if sufficiently justified by a detailed analysis.

6.2.7 Supports on the concrete beams

Prior to concreting, a system of anchor bolts is to be incorporated in the concrete beams, at each location where the trusses are to be supported. The anchor system will receive the trusses on which corresponding baseplates are welded, while the anchor bolts of each location should be connected between each other, with a thin connecting plate, to ensure their distances during concreting (Fig. 6.14).

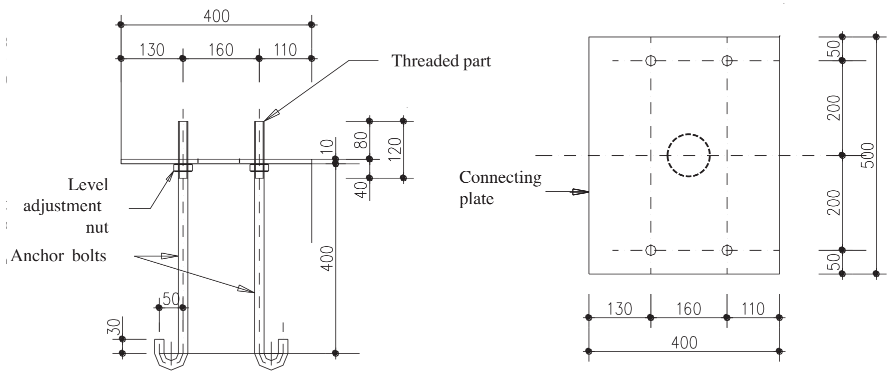


Fig. 6.14. Anchor bolts system in trusses support places

The system is fixed to the reinforcement of the concrete beams with appropriate accuracy concerning levelling and place in plane. Usually the upper face of the connecting thin plate coincides with the upper level of the concrete.

The connecting thin plate could have a round hole at its center to verify that concrete is safely poured under the plate having with it a full contact. Anchor bolts are usually connected to the thin plate through two nuts, one at each side of the plate, providing the possibility of a level adjustment.

To absorb probable small miss of accuracy in the anchor bolts positioning, holes on the trusses' baseplates could be overdimensioned. At the one end of the truss elongated holes should be drilled to absorb thermal elongations.

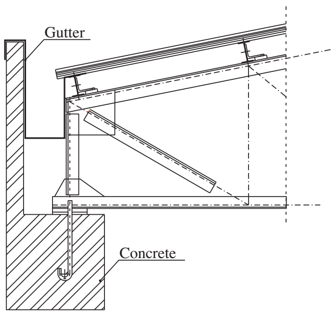


Fig. 6.15. Truss resting on a concrete beam. Gutter detail

In the end detail of the trusses the arrangement for the gutters should be incorporated (Fig. 6.15). Metal gutters are usually made of cold-formed galvanized steel plates, 2 to 3 mm thick.

6.3 Steel framed structures

6.3.1 Introduction

The single span pitched roof symmetrical frame with members (columns, rafters) composed of I- or H- cross-sections is the most common typical frame in industrial, commercial or simple sports steel buildings. For wider buildings additional intermediate series of columns are also provided to reduce the span, in accordance with the functional requirements of the building. For large spans, truss roofs, as discussed before, may be employed to achieve a cost effective solution.

6.3.2 Typical portal frames with members from I- and H- cross-sections

A typical common portal frame is illustrated in Fig. 6.16a. The frame consists mainly of four members (two columns and two rafters), which are assembled on site by bolting and the overall frame will be erected, as a unit, in its final position. This type of frames provides an economic solution for spans up to an indicative dimension of about 25 m. For larger spans alternative solutions with intermediate columns or latticed roofs should be examined.

The frame action is ensured through moment resisting connections between rafters and columns. The maximum value of bending moments under vertical loading appears, for usual frame dimensions, exactly at this connection. Therefore, to obtain an economical cross-section for the rafter, a haunch is usually foreseen at the eaves (Fig. 6.17), providing in addition sufficient space to arrange the required number of bolts. The height of the haunch h is usually smaller than the rafter's depth while its length g is between the 1/20 and 1/10 of the frame's span. The haunch can be fabricated from the same cross-section, as the rafter, and h could result after cutting one

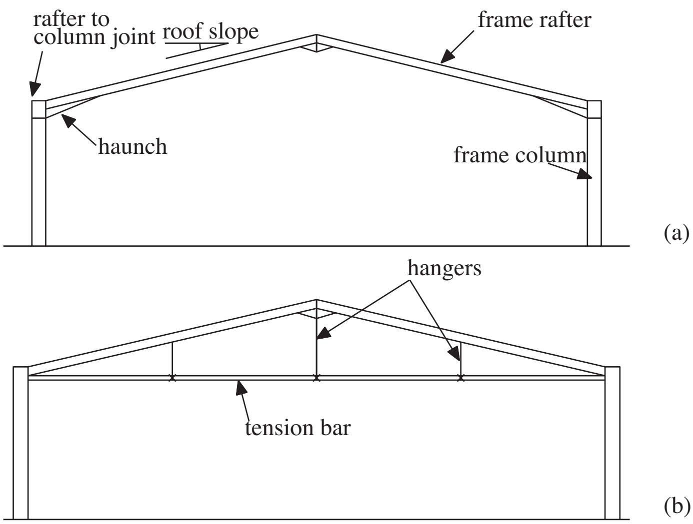


Fig. 6.16. Typical steel frames with I and H cross-sections

flange and the corresponding root radius. Alternatively the haunch may be fabricated from plates. In this way the stiffness ratio I_2/I_1, between the rafter and the column cross-sections, varies usually between 0.25 and 0.50. A haunch may also be provided at mid-span, at the top of the frame (Fig. 6.16a).

The columns may be formed, at their bottom, either as pinned or fixed. Pinned columns are more suitable for unfavorable soil conditions. They lead to the simplest anchor bolts' arrangement and to the smallest concrete dimensions regarding the foundation. On the other hand fixed columns have advantages concerning the distribution of bending moments and the deflections of the frame which might be critical when demanding serviceability requirements exist.

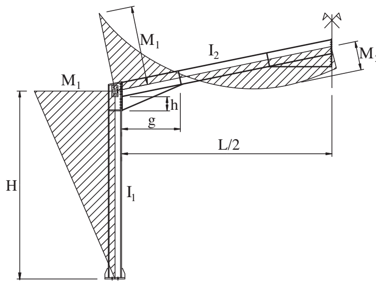


Fig. 6.17. Strengthening eave-haunch in the rafter to column joint

In case of pinned columns members with a variable cross-section could be used (Fig. 6.18a). A possible way to fabricate such a column from hot rolled sections is illustrated in Fig. 6.18b. To reduce bending moments, horizontal reactions at the foundation as well as deflections due to vertical loads, a tension bar may be arranged to join the eave nodes (Fig. 6.16b). This alternative is chosen when cranes are supported by the structure and therefore strict deformation limits are required. In addition, the above arrangement results in reduced bending moments in the columns and allows for smaller cross-sections, which leads to more flexible frames and a possible necessity to perform a second order analysis. Attention should be given to the rafters where important axial compressive forces develop under vertical loading.

The frame members are initially sized according to strength and deformation criteria. During the final design, the stability of the members shall be checked, i.e. their capacity to resist against flexural or lateral-torsional buckling. Concerning flexural buckling, the in-plane buckling length of the columns depends on the H/L and I_2/I_1 ratios (Fig. 6.17) and in case of first order analysis it can be determined using the nomograms for sway frames as presented in 7.2.2. In this calculation, the slopes of the rafters, if of usual values, may be neglected and the buckling length is calculated considering a horizontal rafter with a length L being equal to the sum of the lengths of the two inclined rafters [6.6]. For fixed columns the coefficient of equivalent buckling length β varies between 0.5 and 1.0 (for an infinite rigid or correspondingly an infinite flexible rafter). For pinned columns this coefficient has a value greater than 2.0. For the rafter a buckling length coefficient equal to 1.0 could be conservatively adopted.

The members of the frame should also be checked against lateral-torsional buckling (LTB). The purlins, when connected to the compression flange of the rafters, offer the lateral restraint to exclude LTB phenomena. However, bending moments change sign along the rafter, for the same type of loading. In addition they change sign between different types of loading, such as permanent, live, positive or negative wind loads. The rafter's flanges are therefore partially under compression and par-

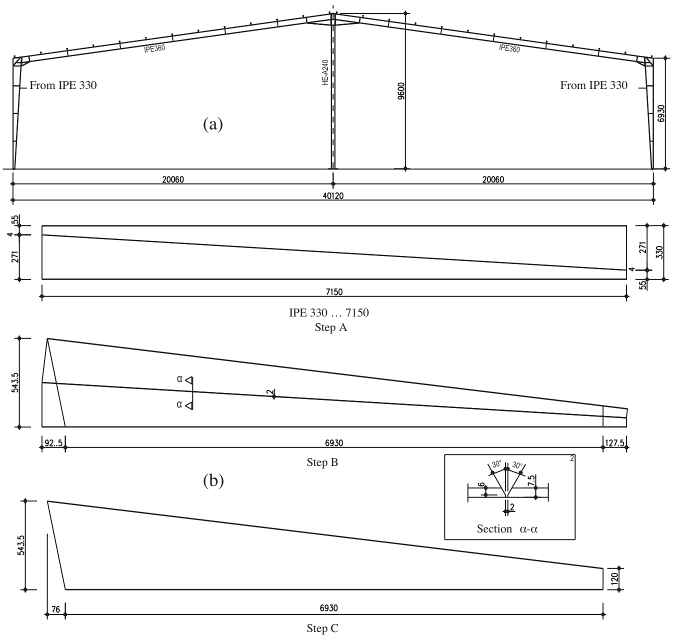


Fig. 6.18. Frame with tapered columns. Procedure to produce such a column from a hot-rolled member

tially under tension, depending on the position along the rafter and the type of loading (Fig. 6.19). When the purlins are used only to transfer loads from the cladding panels to the primary structure, the lateral support to the rafters is provided by the horizontal bracing systems (see 6.4).

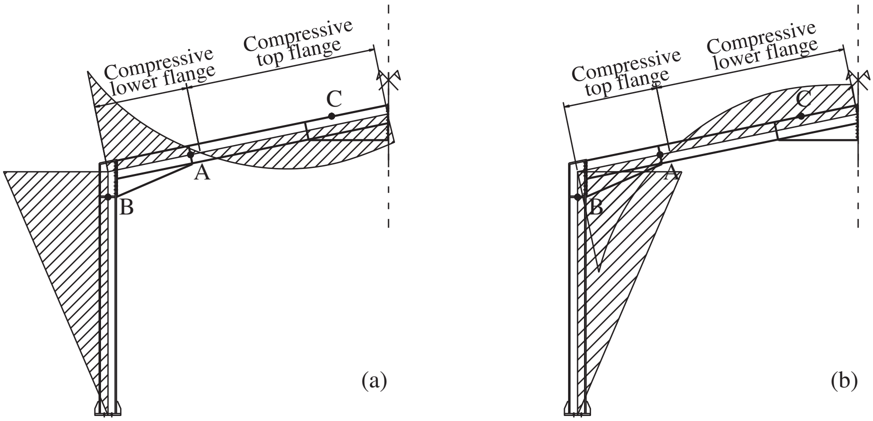


Fig. 6.19. Frame with pinned columns under (a) permanent & live loads (b) uplift conditions. Bending moment diagrams

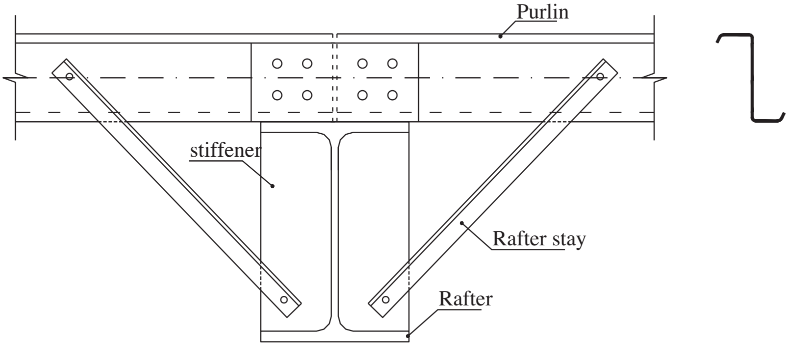


Fig. 6.20. Torsional restraint of a rafter with a lower flange under compression through rafter-stays

Where the compressive flange is not laterally supported, additional elements should be provided to ensure a torsional or lateral restraint in selected cross-sections of the rafter. Such elements could be inclined bars (rafter stays) connecting the compressive flange to the purlins (Fig. 6.20) or longitudinal bars anchored at rigid parts of the overall structure.

When plastic analysis is employed for frame design, the positions where plastic hinges will be formed should be torsional restrained. Such positions are the end of the rafter haunch (point A, Fig. 6.19a) or the top of the column below the haunch (point B). In case of strong and fixed columns, a probable plastic hinge location is the one under the next to the top purlin (point C, Fig. 6.19a) where, in general, the maximum bending moment of the rafter appears. In any case when the unrestrained length of the rafter's compression flange is significant, additional intermediate lateral restraints should be provided. The LTB verifications should be performed in accordance to the methods presented in Chapter 4.

Similar approaches should be followed for providing and checking LTB for columns. The compressive flange at the top of the column, below the haunch (point B, Fig. 6.19a), should be laterally supported by means of inclined horizontal bars to the side rails or through longitudinal bars anchored to the vertical bracing systems. In cases of columns of significant height the necessity of additional intermediate restraints should be examined.

Concerning the formation of plastic hinges, it is reminded that for buildings in seismic regions plastic hinges should be formed in beams and not in columns according to the provisions of Eurocode 8 (EN 1998) [6.7]. As an exception, EN 1998 allows for single storey buildings the development of plastic hinges at the top and the bottom of columns, in cases where the design axial force in the seismic combination is less than the 30% of the plastic axial force resistance of the corresponding column cross-section.

For buildings of significant width, multi portal frames may be employed (Fig. 6.21). Intermediate columns could have moment resisting connections to the rafters or designed as pinned at both ends in order to transfer only vertical loads.

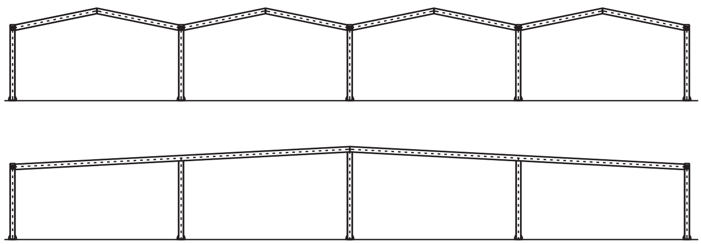


Fig. 6.21. Types of multi openings frames

Horizontal actions are then resisted only by the end columns. Attention, in this case, should be given during erection.

Instead of members from hot rolled cross-sections, members with built-up welded I-sections could also be used, where the cross-section is formed by welding of plates. This gives the possibility to vary the plate thickness, for flanges and web, as well as the cross-section depth, along the length of the member to accommodate the bending moments and shear forces' diagrams, resulting in lighter and more economic frames. This approach is followed by many manufacturers of prefabricated steel buildings. An example of such a frame is shown in Fig. 6.22. Cross-sections in this case are usually classified in categories 3 or 4, therefore only an elastic design may be employed, exploiting the elastic cross-section resistances. For this kind of buildings in seismic regions, only an elastic response is, in general, acceptable.

Frame deformations should be limited to avoid damages to the non- structural elements of the building, noises and users' inconveniencies. According to EN 1990 [6.8] for single storey buildings without cranes, the horizontal displacement at the top of the columns, for the serviceability load combinations, should not exceed H/150, where H is the height of the building. For buildings carrying cranes specific and more demanding limitations should be applied (see clause 6.6).

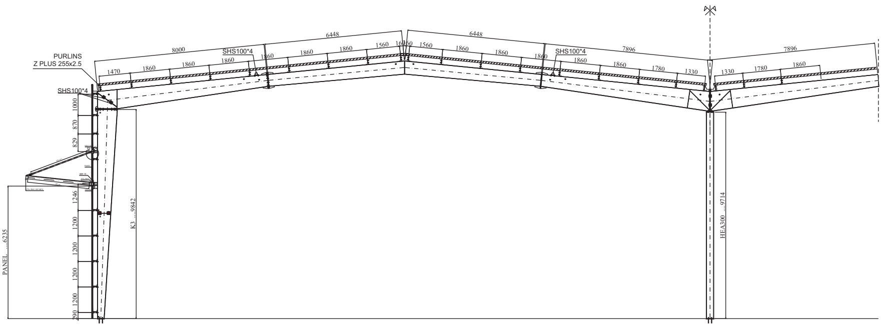


Fig. 6.22. Prefabricated frame with members having variable built-up cross-sections

6.3.3 Frames with trusses as horizontal members

For relatively significant spans (indicatively for more than 25m) typical frames with I- and H- sections for the columns and truss roofs could be employed to obtain a lighter and economical structure. The design criteria for the trusses are similar to those presented in 6.2. Typical forms of such frames are illustrated in Fig. 6.23.

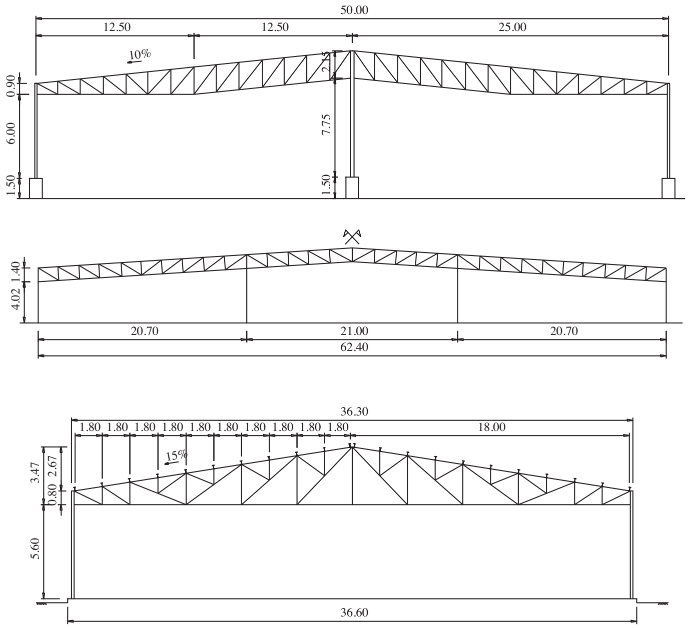


Fig. 6.23. Frames with trusses as horizontal members

Trusses are connected to the columns through gusset plates, in the plane of the column web, using bolted connections to facilitate erection (Fig. 6.24).

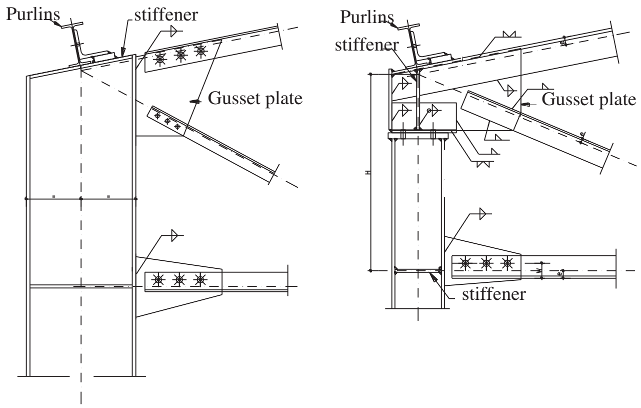


Fig. 6.24. Column to truss connection

As some truss bars of the trusses lower chord could be subjected to compressive forces under both vertical and horizontal loading, attention should be given to provide adequate lateral support in a number of nodes. The web of the columns should be provided with transverse stiffeners at the level of the truss chords. The constructional detail of the connection between truss and column should be such to allow sufficient space for the gutters to be provided (Fig. 6.25).

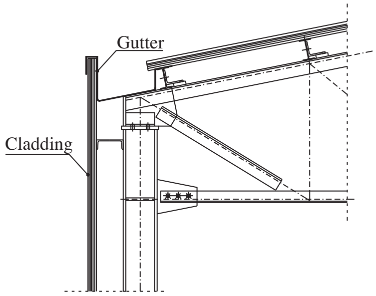


Fig. 6.25. Gutters arrangement

6.3.4 Single storey buildings with operating cranes

A common form of a frame for a single storey building, with one (or more) operating crane bridge, is shown in Fig. 6.26. Columns are provided with sufficient strength and rigidity, up to the level of the crane bridge, in order to avoid deformations beyond the limits required by serviceability criteria (see 6.6.4). They are usually formed as built-up columns with two main flange members connected with bracing bars or batten plates. Typical examples of such buildings are shown in figures 6.27 and 6.28. The top flange of the runway beam may be laterally supported by a horizontal surge element, such as a beam or a lattice girder, in order to protect it against lateral-torsional buckling, to limit horizontal deformations and transmit crane surge to the supports.

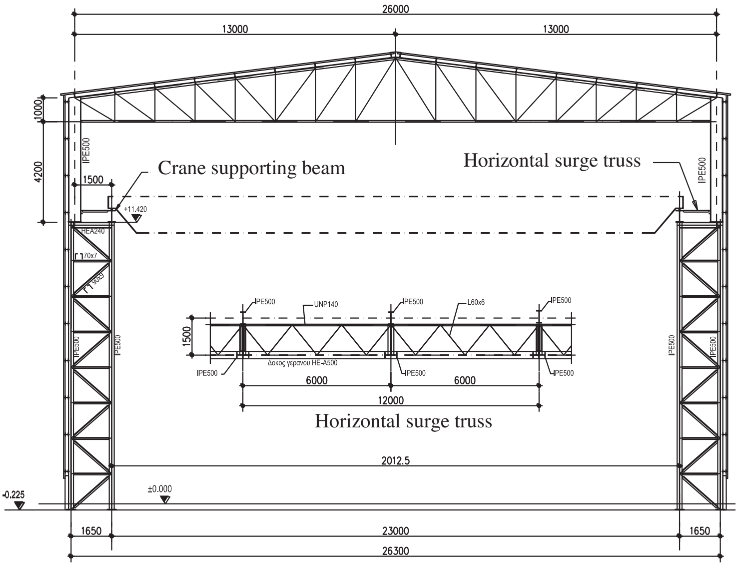


Fig. 6.26. Typical frame of a building with a top mounted crane

Such a support element is also shown in Fig. 6.26 and could additionally be used to support a catwalk along the building, helpful for inspection and maintenance purposes. The top of the internal column, that supports the runway beams, should be restrained against out-of-plane deformations. The roof structure could be a truss or a rafter, depending on the span and the designer's choices.

For cranes with relatively small hoisting capacity, a single column could be used while the crane bridge would be supported by short cantilevers welded to the columns (Fig. 6.29). These welds should be carefully executed and verified against fatigue. Tension bars, as discussed before, at the eaves level, may be provided to reduce deflections at least for the vertical loads. The deformation criteria for such buildings are presented in section 6.6.

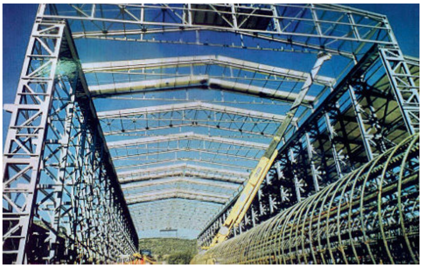


Fig. 6.27. Shipyard building with laced built-up columns up to the level of the runway

Fig. 6.28. Shipyard building having built-up columns with batten plates up to the level of the runway beams

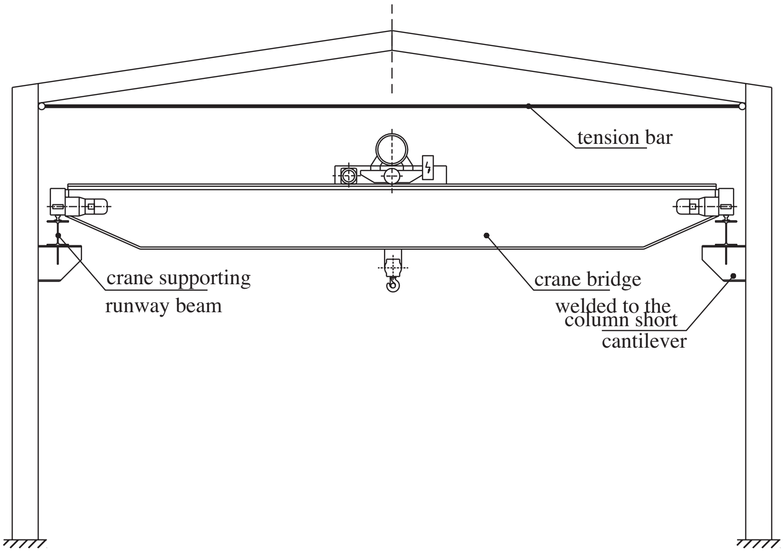
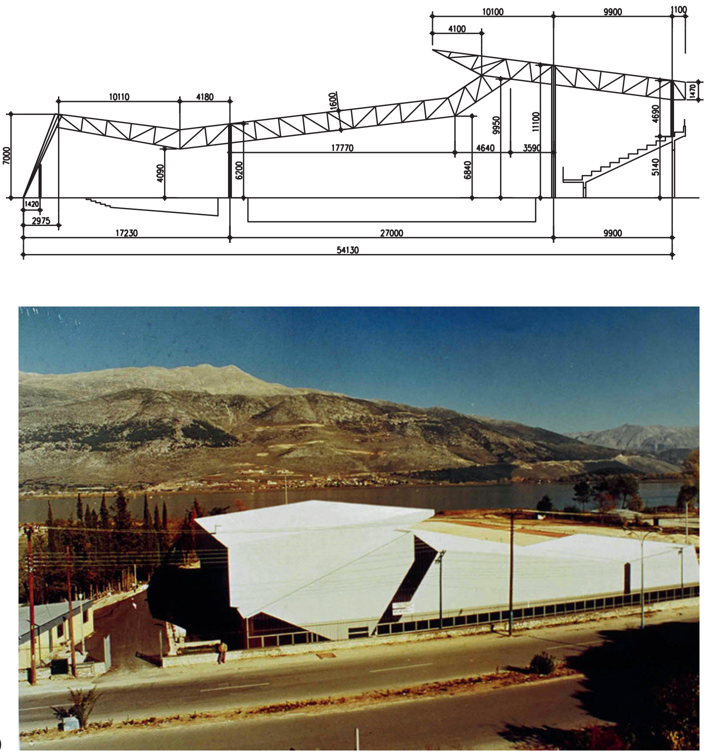


Fig. 6.29. Runway beams on short cantilevers in a typical frame

6.3.5 Buildings of complex geometry or for special purposes

The practice of using repetitive plane frames according to the one main direction of a building and bracing systems according to the transverse one, could also be applied for buildings with a more demanding architectural elaboration or functional lay-out. Examples of such building and the corresponding plane frames are presented in Figure 6.30 for the roof of a swimming pool, in Figure 6.31 for the roof of a sports center, in Figure 6.32 for an air planes hangar and in Figure 6.33 for the covering of soil material to be used in a cement industry.



(a)

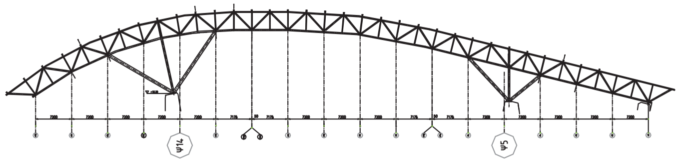
(b)

Fig. 6.30. Typical frame (a) of a swimming pool building (b)

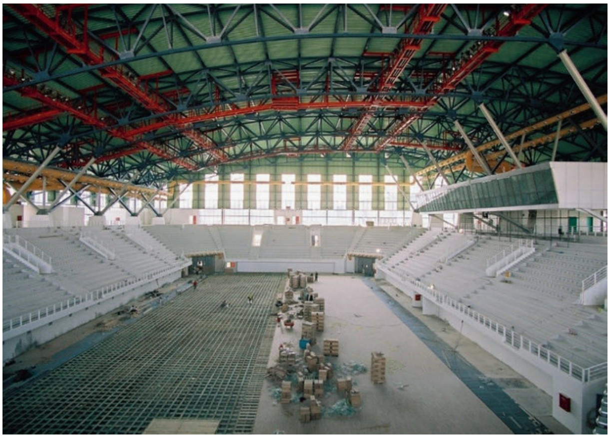
6.3.6 Anchorage

6.3.6.1 Concrete foundation

Concrete foundations in single storey steel buildings are subjected to small vertical and, in general, to relatively substantial horizontal forces (due to wind, cranes, thermal expansions, seismic actions). Therefore the foundation should be carefully

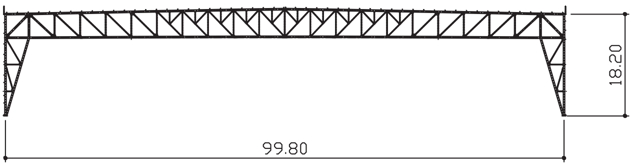


(a)



(b)

Fig. 6.31. Typical frame (a) in a sports centre (b)



(a)

(b)

Fig. 6.32. Typical frame (a) in an airplanes maintenance hangar (b)

checked against overturning and sliding. In this verification the self-weight of the foundation is an important factor which provides stabilizing forces and reduces the resulting eccentricities. Foundations consist mostly of single concrete blocks, under

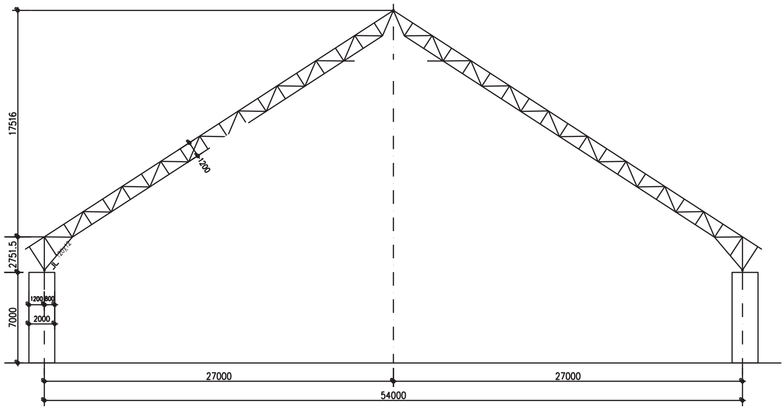


Fig. 6.33. Typical frame of a roof covering soil material

each column, joined together by connection beams (Fig. 6.34a) which could also be used for the encasement of the filling compacted soil material. As an alternative a continuous foundation beam could be constructed along each column line.

Horizontal forces, transverse to the main frames direction, may have significant values especially in sides where vertical bracings between columns are arranged. In this case a unified foundation could be provided for both columns of the bracing panel (Fig. 6.34a, b). When there is a distance between the upper level of the foundation block and the functional level of the building, short concrete columns are provided to reach the level of the steel columns' baseplates (Fig. 6.34b).

6.3.6.2 Anchor bolts

The typical anchoring procedure starts by incorporating, before concreting, into the foundation a system of anchor bolts. The anchor bolts are interconnected with relatively thin plates to ensure that distances remain unchanged. Nuts placed under the plates allow final leveling. The upper surface of this plate coincides usually with the upper concrete surface (Fig. 6.35). Anchor bolts are often subjected to substantial tensile forces, such as in fixed columns due to bending moments or in columns participating in the vertical bracing systems. Anchorage of the bolts, in case they are fabricated of mild steels, could be provided by hooks or, more usually, by anchor plates (Fig. 6.35). Uplift anchor forces are resisted by a conical surface of concrete. To ensure a safe connection to the foundation, especially in cases of repetitive or dynamic loads, a double nut system for the anchor bolts is very often used. The latter could also be placed into tubes, allowing in this way for the absorption of small geometric deviations during erection (Fig. 6.36).

In order to arrange the final leveling of the baseplates, a bedding space of about 25 to 70 mm is left between them and the foundation level which is filled, after final leveling, usually by a no shrinkable mortar or by other types of cement based grouts or fine concrete. For baseplates of large dimensions holes are provided in the plates to ensure that a full contact between them and the foundation is realized. As, in such a case, a small distance exists between the foundation and the shear plane of the

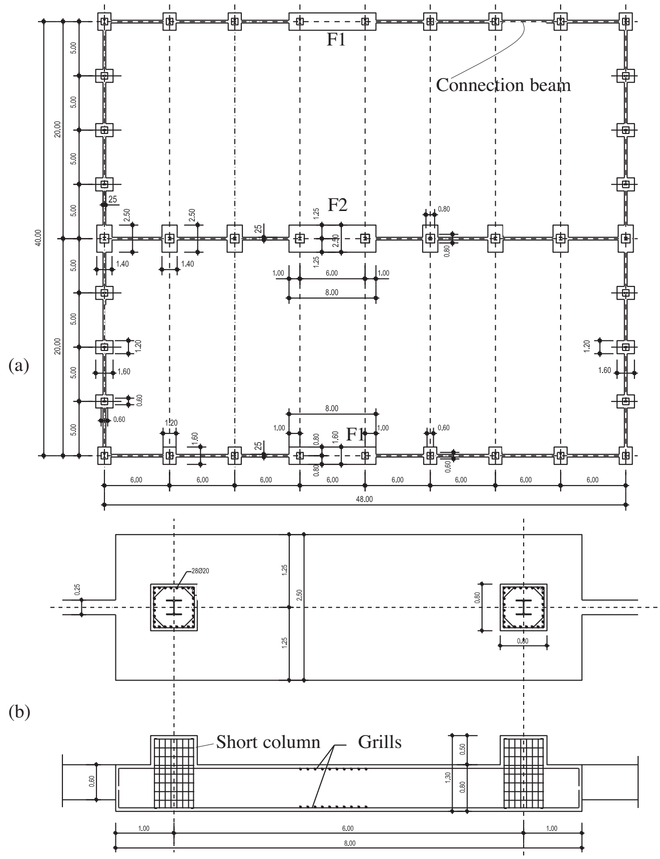


Fig. 6.34. (a) Foundation plan of a steel building and (b) detail of foundation block

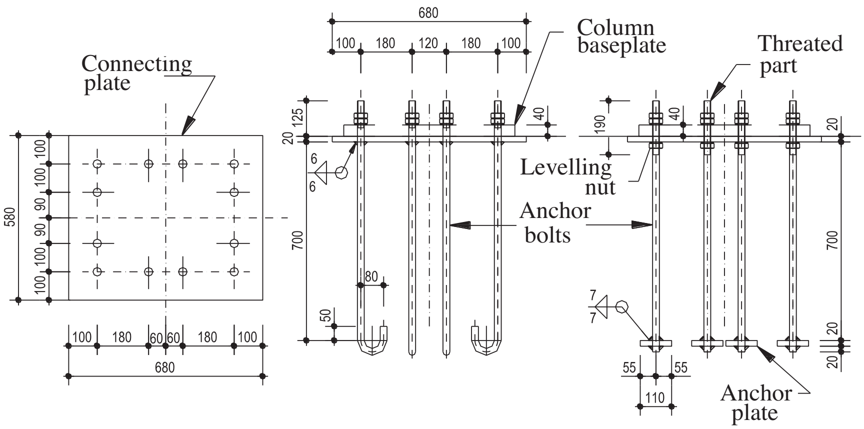


Fig. 6.35. Typical system of anchor bolts assembled through a connecting plate. Anchorage using hooks or anchor plates

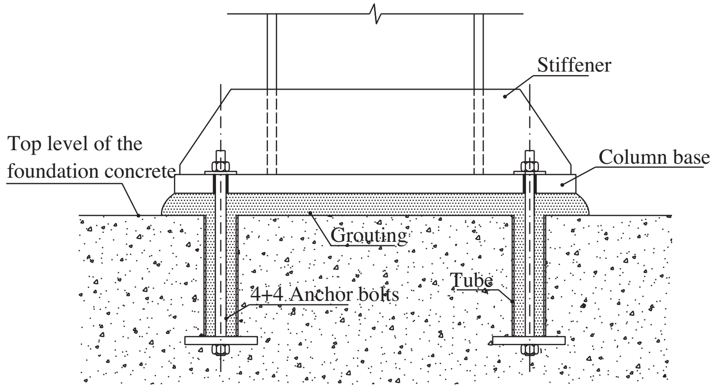


Fig. 6.36. Fixed column base. Anchor bolts placed in tubes

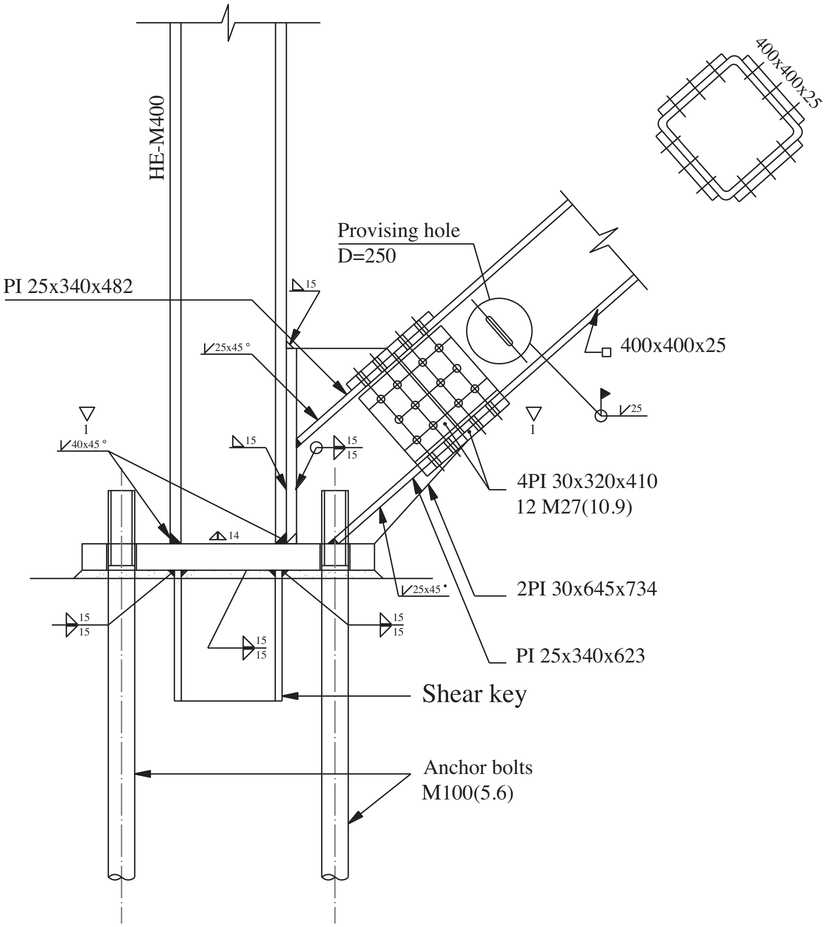


Fig. 6.37. Baseplate with important horizontal reaction and a shear key

anchor bolts, a stiff element, called “shear key”, such a short beam, is usually welded on the lower surface of the baseplate to resist shear forces and to avoid shearing and secondary moments in the bolts (Fig. 6.37).

In smaller and simpler buildings the foundation blocks could be constructed with pockets (Fig. 6.38) to avoid anchor bolts installation and to allow for geometric deviations absorption, according to both vertical and horizontal axes. Pockets will be filled with a second phase dense concrete, having a compressive strength at least equal to the one of the already constructed foundation. The depth of the pocket and the embedded column length depend on the value of the bending moment, at the column base, and may be such that stresses σ_b in the concrete (Fig. 6.38) remain within acceptable limits. During erection the column’s stability could be ensured by provisory auxiliary steel elements or by filling the pocket with concrete to a sufficient length. In the second case it is considered that stability is ensured when this concrete gain at least the half of his intended strength.

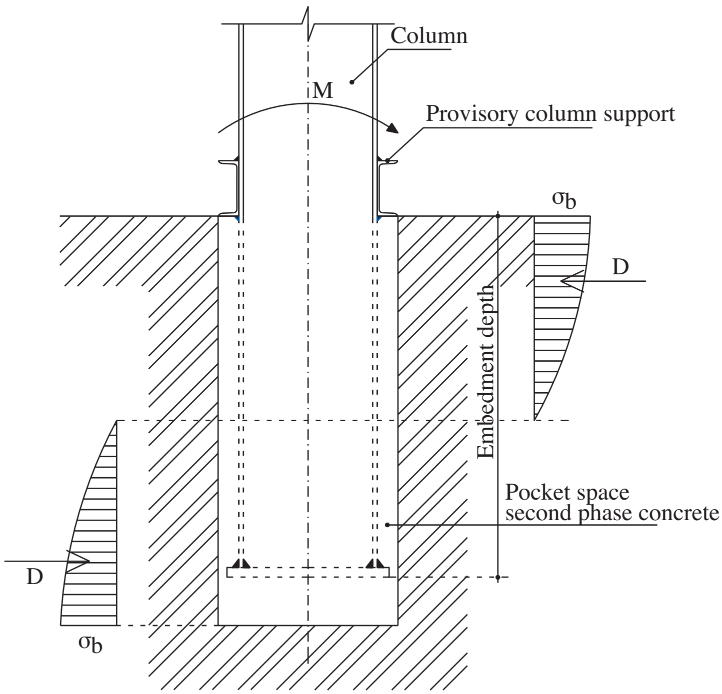


Fig. 6.38. Foundation block with pocket space

6.3.6.3 Fixed and pinned column bases

For fixed columns, a system of anchor bolts around the column should be arranged to provide adequate lever arms. Due to bending moments, anchors on one side of the column are subjected to tension forces while the baseplates to bending. To limit these bending moments anchor bolts should be as close to the column flanges is

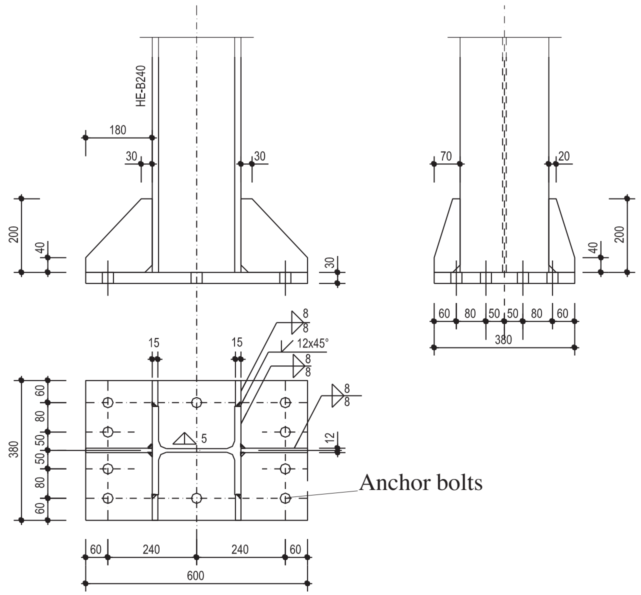


Fig. 6.39. Stiffened baseplate of a fixed column

possible and provide plates with stiffeners by which excessive thickness is avoided (Fig. 6.39). When the depth of the foundation block is not sufficient, anchor bolts may be connected with horizontal bars incorporated into the concrete in order to transfer anchor forces to the concrete not by shear but by contact through these bars (Fig. 6.40). Fixed column bases are totally ensured against rotation only for foundation in rock. For foundation in soil there is always a rotation as the soil is subjected to small subsides, leading to a certain rotation. For weak soil conditions a fixed column base could be realized only after a local soil improvement or by using piles.

For pinned column bases a limited (usually two or four) number of anchor bolts is needed, arranged close to the axis of the cross-section, in the area between flanges. Two bolts on the cross-sectional axis is closest to the assumption of simple connection used in the analysis while four bolts (nominally pinned joint) can ensure a better column stability during erection. Pinned column bases require smaller foundations.

6.4 Bracing systems of the building

6.4.1 Introduction

Bracing systems in a building are mainly provided to resist horizontal forces acting transverse to the main frames and to transfer them to the foundation. In addition they offer lateral support to the aforementioned main elements and they play an important role during erection. They are divided into horizontal bracing systems, arranged between successive rafters of frames or top chords of trusses and vertical bracing systems arranged between columns.

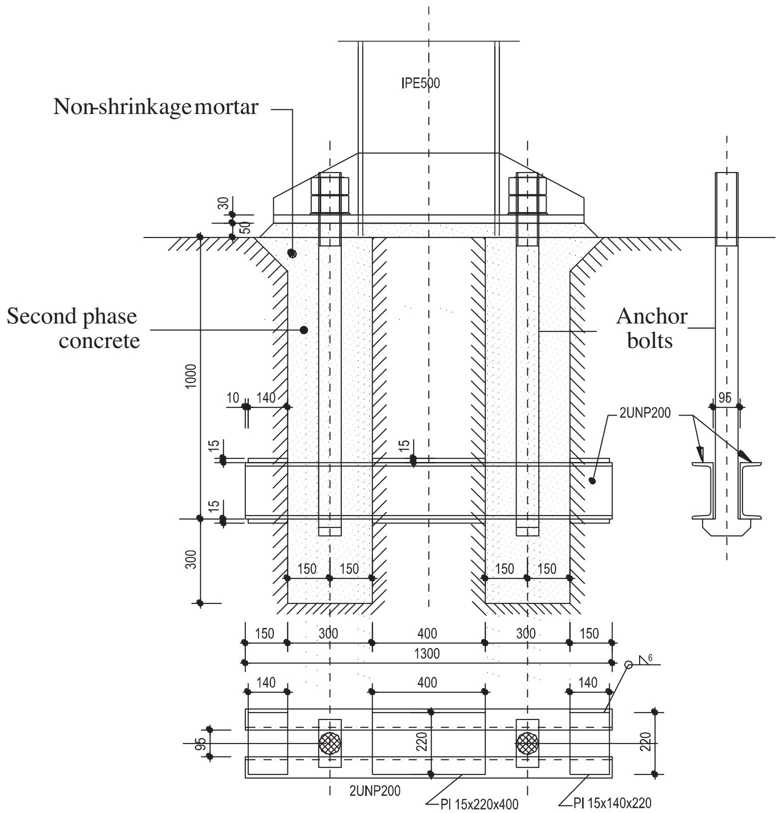


Fig. 6.40. Anchorage to horizontal bars incorporated to the foundation

6.4.2 Horizontal (or wind) bracing systems

Horizontal, or wind, bracing systems are, in general, arranged at the roof of the building. They transversely connect the main plane frames to a complete 3D main structure and provide a diaphragm action in the roof. Its elements are placed at the level of the upper flange of the rafters with I- or H- cross-sections or at the level of top chords in the case of trusses. Horizontal bracing systems connect only a few main frames being usually placed in the end panels (first and last) of the building and at intermediate locations every 4 to 6 panels. Together with the purlins, or alternatively with the aid of additional bars, they provide lateral support to all main frames, not only to those they directly connect. Horizontal bracing systems are composed of diagonal bars, while the role of posts may play the purlins or additionally arranged bars.

When the purlins are part of the bracing system, diagonal bars are placed between purlins (Fig. 6.41a). The truss bracing system consists of the two adjacent rafters, the diagonal bars and the purlins behaving as posts (Fig. 6.41b), where in the analysis model it is assumed that effective are only the tension diagonals (Fig. 6.41c).

The diagonals are bolted to gusset plates, which are welded to the top flange of the rafters (or the top chord of trusses), and at their middle to the intermediate purlins (Fig. 6.42). This type of horizontal bracing, with participation of the purlins, is applicable for purlins from hot rolled profiles. The specific purlins that are part of the bracing system, beyond their main role to resist vertical loading, may have a stronger cross-section keeping the same depth with the typical ones. For the diagonals equal leg angles are commonly used or, alternatively, circular hollow cross-sections or wires. The end diagonals are the ones stressed the most, but usually the same cross-section is used for all of them. Angles are bolted on one leg, a fact that should be considered in the design as a tension element (see 3.2).

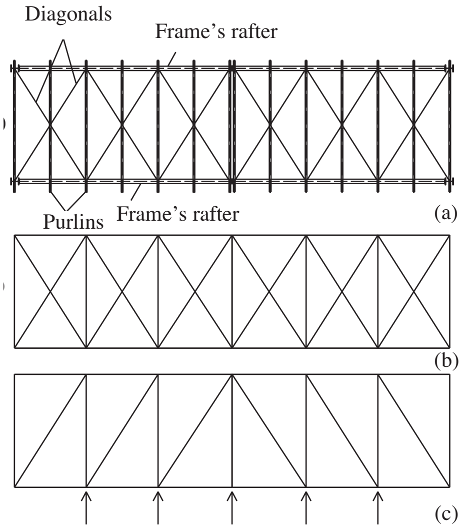


Fig. 6.41. Horizontal bracing system between rafters

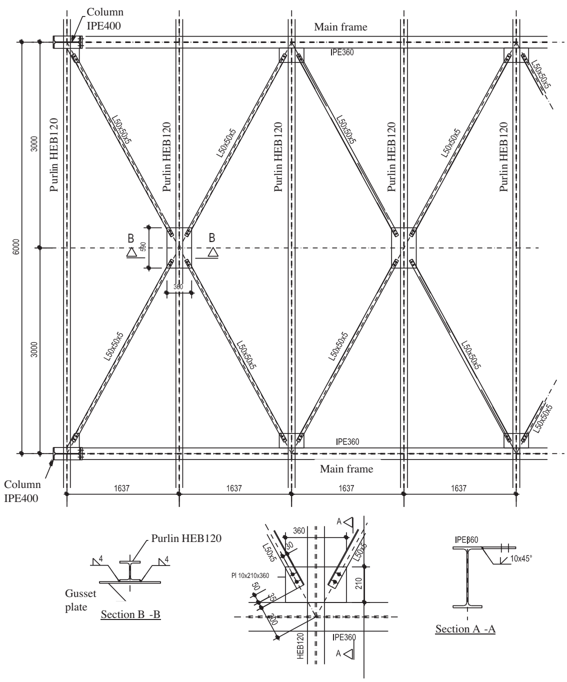


Fig. 6.42. Horizontal bracing system with the participation of purlins. Details

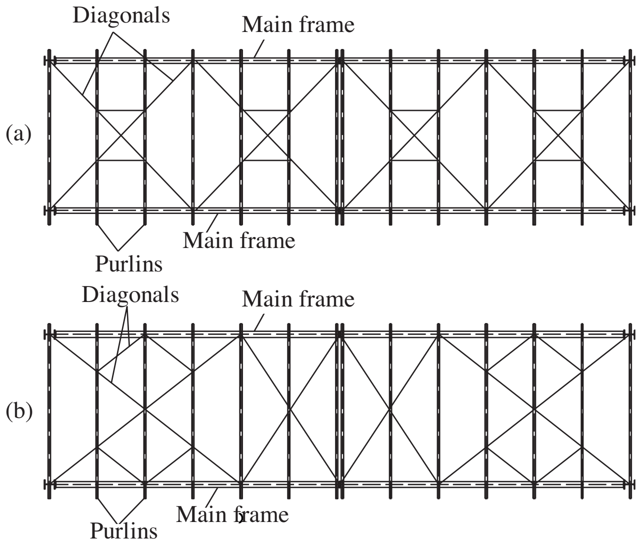


Fig. 6.43. Horizontal bracing system with diagonals covering (a) three or (b) four panels between purlins

To avoid small angles between diagonals and purlins, diagonals could be extended to connect three (or more) purlins (Fig. 6.43a, b).

When cold-formed thin-walled purlins are employed, an independent bracing system is provided without the participation of the purlins. In such cases the level of the bracing system is usually at the level of the rafter's web, nearer to the upper flange, in order to limit the eccentricity between bracing system and the cladding surface, to offer a more efficient lateral support to the rafters and to allow for the placement of bracing bars and connections (Fig. 6.44).

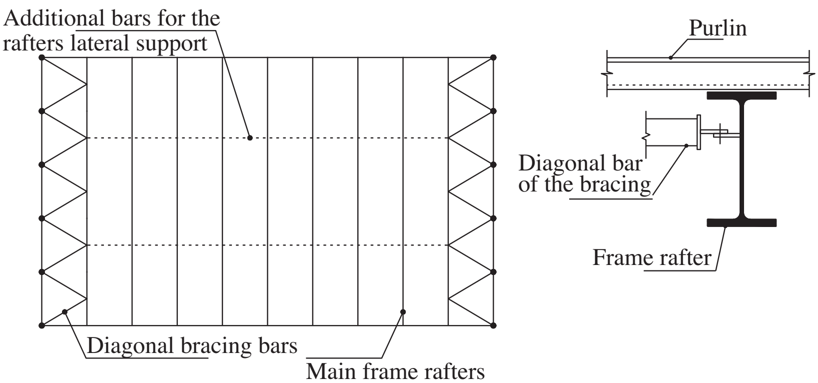


Fig. 6.44. Independent to the purlins horizontal bracing system

Alternative bracing geometries may be employed with bars resisting both tension and compression. An indicative form is shown in Fig. 6.44. Circular hollow sections are commonly used for the bracing bars allowed to resist compression. Additional longitudinal bars may be needed to support laterally intermediate frames (Fig. 6.44).

As already mentioned horizontal bracing systems resist horizontal forces acting in the longitudinal direction of the building and offer lateral support to the main frames. Eurocode 3 offers an application rule to determine additional lateral forces that bracing systems have to resist for providing the above lateral support as a function of the number of the frames they laterally stabilize (see 2.13).

In addition to the horizontal bracings described above, it is a good practice to arrange horizontal X-bracings along the building, close to the column axes (Fig. 6.45a). This supplementary bracing can distribute any horizontal action applied to a specific frame (for instance a crane horizontal action) to the adjacent ones, limits differential horizontal displacements and offers a better diaphragm action in the roof. In buildings where in an intermediate column line some columns are missing and therefore axes with different rigidities exist (Fig. 6.45b), the above supplementary bracings improve also the diaphragm action.

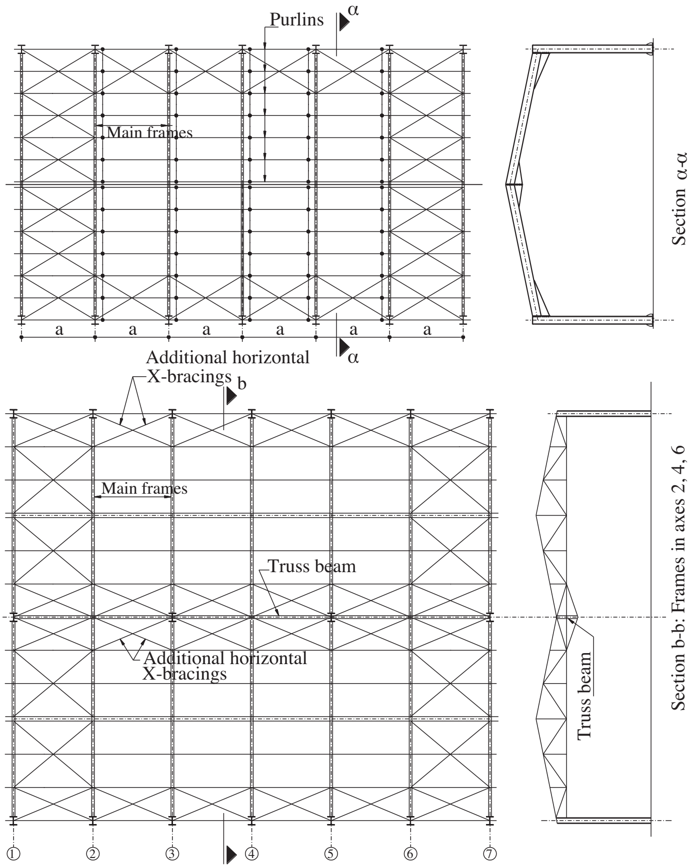


Fig. 6.45. Additional lateral X-bracings

6.4.3 Vertical bracing systems

Vertical bracing systems are formed by adding diagonal bars between columns in selected panels, in order to increase significantly their rigidity. These systems resist horizontal actions arising from the horizontal bracings and transfer them to the foundation. In addition they form the rigid structural part for the anchorage of longitudinal elements such as bars or side rails which offer, when needed, lateral support to the columns and ensure overall stability during erection. For the diagonal bars hollow sections (circular or square) or built-up sections are usually selected. The required cross-sections' capacities depend mainly on the magnitude of horizontal forces and whether the building is in seismic areas or not.

An important element of the vertical bracing system is the eave strut that joins the columns at their top and distributes horizontal forces to the various vertical bracings. The eave struts are elements mainly subjected to compression and therefore should have an appropriate cross-section. They are usually connected to the column webs through simple bolted connections (Fig. 6.46).

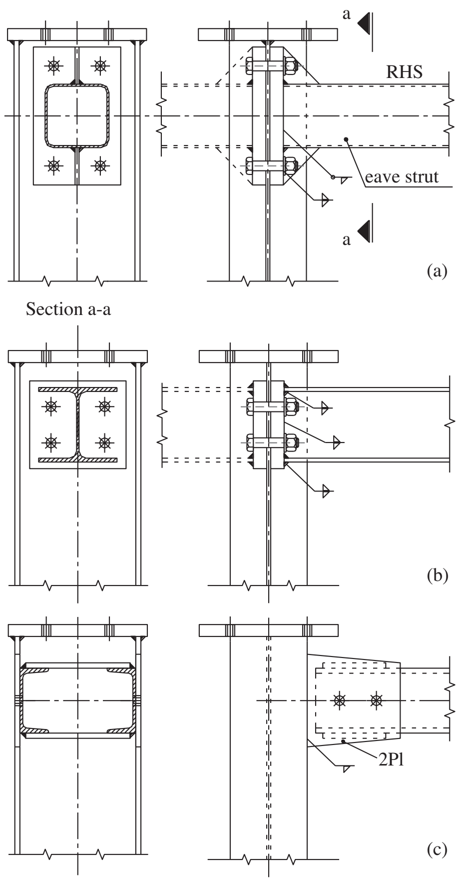


Fig. 6.46. Eave struts cross-sections. Connection to the columns

Instead of X-bracings other types of truss systems may be applicable, as it is indicatively shown in Fig. 6.47, depending on the necessity for openings or for a free access, and the requirements of the seismic analysis, if any. In case of buildings with significant height the bracing system may be composed of two or more sub-panels with intermediate horizontal members (Fig. 6.47 e, g) to avoid small angles between the diagonals and the other structural members of the bracing such as columns, eave struts or intermediate horizontal bars, as well to avoid diagonals with a substantial length. Intermediate horizontal elements could extend along the whole building length, offering to the columns lateral support, if needed, in respect to the out-of-plane buckling.

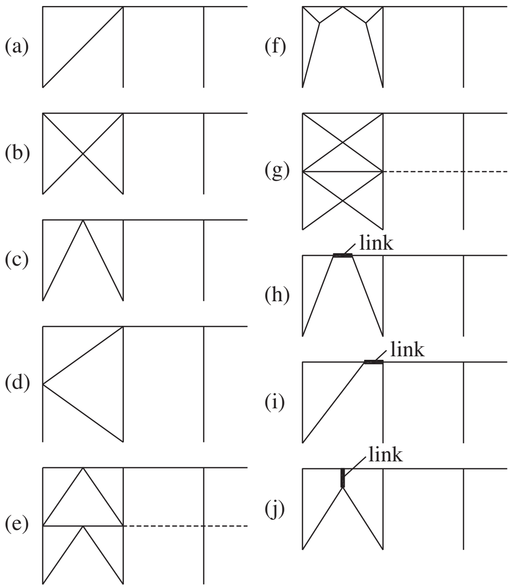


Fig. 6.47. Types of vertical bracings

When the introduction of diagonal bars is not possible for functional reasons, a frame action, in the transverse to the main frames direction, could also be provided. In such a case the eave beams, all or some of them, are to be connected to the columns through moment resisting connections. Alternative arrangements could also be adopted, as for instance to insert in some panels relatively rigid secondary frames (Fig. 6.48).

The vertical bracing systems are mostly usually arranged in the same panels as the horizontal bracings which are generally advised to be arranged in the end panels. However stiff vertical bracing systems at the ends of the building restrain thermal expansions and create significant forces which should be considered in the analysis. As an alternative, vertical bracings could be arranged at intermediate panels (preferably

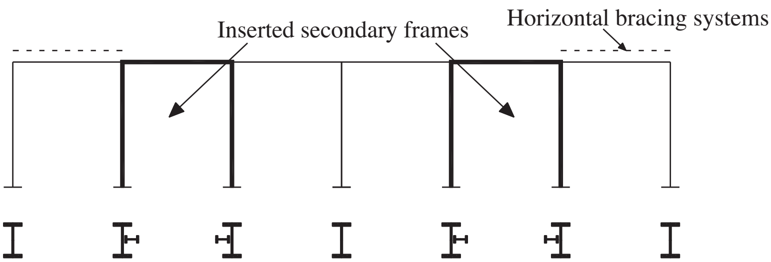


Fig. 6.48. Inserted secondary frames in selected panels between columns

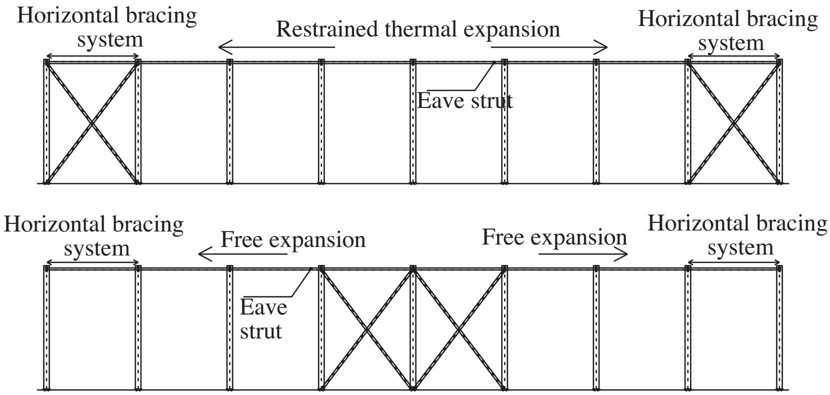
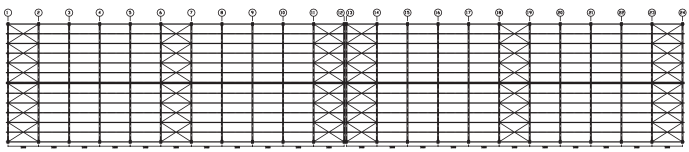


Fig. 6.49. Alternative arrangements for the vertical bracings

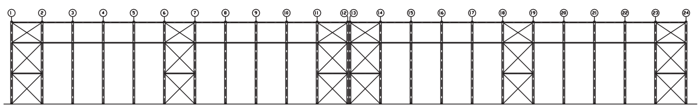
at the middle of the building, see Fig. 6.49). In this case thermal forces are reduced and horizontal actions are transferred to the vertical bracings through the eave struts. In all cases the erection starts from the panels where vertical bracings are located.

Vertical bracing systems could be divided in concentric bracings, in which horizontal actions are resisted by members subjected to axial forces, and in eccentric bracings where the forces are mainly resisted by axially loaded members and in addition by bending of other members resulting by the eccentricity between bracing bars. In specific cases horizontal forces could be resisted by a combined system of moment resisting frames with concentric bracings or by cores of concrete or concrete walls.

When expansion joints are not provided in steel buildings significant thermal forces may develop dependent on the building length. When such joints are arranged, in the axis of the expansion joint usually two different frames are provided, that divide the building in statically independent structures (Fig. 6.50). In buildings with significant dimensions, optimal places of the thermal joints should be selected. The clear width of the joint is to be defined considering expansion, constructional and seismic criteria.



(a) Plan view



(b) Lateral view

Fig. 6.50. Expansion joint in a building of large length

6.4.4 The seismic behavior of the vertical bracings

6.4.4.1 General remarks about seismic design

Among horizontal forces resisted by the vertical bracings, seismic forces constitute a specific part of them, their importance being significant especially in regions of high seismicity. Seismic design is oriented towards the arrangement of specific structural zones or members which, in case of a seismic event, have the possibility to absorb part of the seismic energy through the development of cyclic plastic deformations and therefore to limit seismic forces to the other structural members. In the above zones or members, called dissipative zones (or dissipative members), damages might appear after a strong seismic event, so that dissipative members should have the possibility to be replaced after the earthquake. Steel, due to its capacity to develop important plastic deformations, before fracture, is an ideal material for such an approach in the design of a structure.

Having in mind the above considerations the seismic behavior of vertical bracings is valuated according to their capacity to develop, in some members, such plastic deformations. To this way, types of bracing systems associated with a brittle type of fracture such as flexural or torsional buckling, brittle failure of tensile members, failure of bolts due to shear or tension, are not ideal for seismic areas. In this category are classified V- and especially K-type bracings (Fig. 6.47 c, d). Other types of concentric or eccentric bracings are considered to have a more efficient seismic behavior.

In concentric bracings the dissipative zones are mainly located in the tensile diagonals. The most common type of such bracing is the X-bracing (Fig. 6.47b, g) with active tensile diagonals, where horizontal forces can be resisted by the tensile diagonals only. In eccentric bracings (Fig. 6.47 h, i, j) the dissipative members are the links (horizontal or vertical) which can develop cyclic plastic deformations either due to bending or shear.

It is obviously very important that dissipative members yield first, during an earthquake, before the other elements of the bracing leave the elastic range. To this end, all members, apart the dissipative ones, as well related connections, are designed to have sufficient over-strength. The rules to ensure that dissipative zones yield first, while the other bracing elements remain elastic, constitute the so-called "capacity design". In this design consideration, actual values of materials' properties for the dissipative members, such as the yield stress, are considered, which could be significantly greater than the nominal ones. In many cases dissipative members are provided from a mild steel of lower yield strength compared to the overall structure (for instance dissipative members may be of S235 steel while the overall structure is from S355 steel).

The acceleration in the structure during an earthquake is, in general, defined in the Codes according to an elastic response spectrum in which the influencing parameters (peak ground acceleration of the region, soil conditions, vibration period of the structure, structural damping) are taken into account. In this spectrum it is assumed that the structure's response is totally elastic. The capacity of the structure to resist seismic actions in the nonlinear range, by developing important plastic deformations in the dissipative members, provides the possibility to consider design seismic forces smaller than those corresponding to the above linear elastic response. The above capacity is taken into account in the structural design, by introducing a behavior factor q, greater than unity, which divides the values of the aforementioned elastic spectrum, producing the design spectrum used in the analysis. The behavior factor is an approximation for the ratio between the seismic forces that would be developed following an elastic and an elastic-plastic structural behavior.

6.4.4.2 General rules of Eurocode 8

Design of structures for earthquake resistance is the object of Eurocode 8 (EN 1998). Its part 1 [6.9] provides general rules, seismic actions and rules for buildings. In the present chapter 6 specific rules for steel structures are included while in chapter 7 additional rules for multi-storey steel or composite steel-concrete buildings are presented.

According to the Code one of the following concepts should be chosen: (a) design according to a low dissipative structural behavior (low ductility class, DCL) or (b) a dissipative structural behavior divided in two subclasses, DCM (medium dissipative class) and DCH (high dissipative class). For each design concept the range of applicable q-values is given while the maximum acceptable values of the q-factor, for each type of dissipative system are specified.

When a dissipative structural behavior is chosen, some additional rules related to capacity design should be followed to ensure that yielding will take place first in the dissipative zones. In regions of low seismicity the DCL concept may be chosen without respecting the above additional rules. In this case it is recommended to use, as maximum, a behavior factor $q = 1.50$ which expresses the existing dormant ductility of an overall steel structure.

For X-bracings (Fig. 6.47b, g) in which the active diagonal under tension is the dissipative member, the upper limit for the behavior factor is q=4 (both for the DCM and DCH concepts). For V-type bracings (Fig. 6.47c), in which both diagonals are active and their intersection lies at a horizontal continuous member, the upper limit for q is $q = 2$ for the DCM concept and $q = 2.5$ for the DCH approach. K-bracings (Fig. 6.47d), in which diagonals intersect on a column, should not be used due to the possible formation of a plastic hinge in a column that would endanger structural stability. For eccentric bracings in which the links (Fig. 6.47h, i, j) are the dissipative members, the maximum recommended value of q for the DCM concept is equal to 4. The same value is to be used for moment resisting frames. For eccentric bracings as well as for the moment resisting frames, higher values for q, in a DCH approach, could be applied under additional requirements. Different q-values may be used according to the two main directions of the building, when different structural systems are used to resist seismic actions.

The main general rules in EN 1998 concerning seismic design in general and, additionally, capacity design are as following:

a) Tensile dissipative members must comply with the ductility criterion, as defined in equations (3.13) and (3.14).
b) Sufficient local ductility should be available by the dissipative members, depending on the ductility concept applied and the value of the behavior factor used.

In this frame, for DCL concept and q values between 1.5 and 2, cross-sections of classes 1, 2 or 3 may be used. In the case of the DCM concept and q values between 2 and 4, only cross-sections of classes 1 and 2 should be used. For DCH concept and q values higher than 4, exclusively cross-sections of class 1 shall be used.

c) Cross-sections and member resistances should be checked according to EN 1993.
d) To avoid undesirable effects due to an actual yield stress of dissipative members, larger than the nominal one, an over-strength factor γ_{ov} equal to 1.25 should be introduced in the capacity design.
e) Dissipative zones can be located either in structural members or in connections. However, it is widespread practice that members are used as dissipative zones. Energy dissipation in the connections is still under investigation.
f) When dissipative zones are located in members, the non-dissipative parts of the seismic resistant system as well as the connections of dissipative members at their ends shall have sufficient over-strength to allow for the development of cyclic yielding in the dissipative member.
g) Non-dissipative connections of dissipative members made by means of full penetration welds may be deemed to satisfy the over-strength criterion.
h) In connections with fillet welds or bolts, the required resistance of the connection should be at least equal to 1.10 γ_{ov} R_{fy}, where R_{fy} is the plastic resistance of the connected dissipative member.
i) The shear resistance of bolts in the connection of a dissipative member should be at least 20% larger than the corresponding bearing resistance.
j) For bolted connections in shear, only slip resistant joints of categories B and C (see chapter 5), with preloaded bolts, should be used, while for bolted connections in tension, only joints of category E.
k) In slip resistant connections, only surfaces of categories A and B (see chapter 5) should be specified.

Structural steel used for structures in seismic areas should conform to standards refer to EN 1993. The maximum value of the active yield stress of steels used in dissipative members should be specified in the project specification and noted in the drawings.

Material toughness decreases when temperature decreases and failure modes change from ductile to brittle. In cases where structures or isolated structural elements could be exposed to low temperatures, the influence of the steel and welds toughness variation should be considered in the design. To this end the lowest temperature adopted in combination with the seismic action should be also indicated in the project specification. Recommendations to consider toughness variation in low temperatures, not only under seismic actions, are included in EN 1993-1-10 [6.10].

It is a good practice that horizontal bracing systems have, in seismic design, sufficient over-strength against vertical bracings to ensure that horizontal seismic action is uniformly distributed to them. To this end an over-strength factor of 1.30 could be applied.

6.4.4.3 Specific rules for concentric bracings to Eurocode 8

As already explained, concentric bracings should be designed so that yielding of the diagonals in tension takes place before the failure of the connections and before the yielding or buckling of the beams or columns. In addition to the general rules, as presented above, some additional specific rules for the concentric bracings are recommended in EN 1998:

a) Under gravity loads it is assumed that diagonal bars are not active.
b) In an elastic analysis of a structure with X-bracings, only the diagonals under tension are to be taken into account to resist seismic actions. However both diagonals may be introduced in the analysis when supplementary requirements are fulfilled. In frames with V-bracings both diagonals (tensile and compressive) are to be considered.
c) The relative slenderness of the diagonals should be within a specific range of values. The application rule is

$$1.3 < \bar{\lambda} < 2.0 \tag{6.1}$$

The lower limit of the slenderness is recommended to avoid overloading of the columns in the pre-buckling stage, in which both diagonals are active, beyond the action effects on the columns at the ultimate limit state, where only the tension diagonal is active.
On the other side very slender diagonals buckle early, developing large lateral elastic deflections, and produce weak hysteretic loops during the cyclic seismic action. To obtain sufficiently large loops and therefore to have the possibility of large energy dissipation, an upper limit for the slenderness is recommended.
In V-bracings only the upper limitation applies.
In structures having up to two stories, the above limitations for the slenderness can be omitted.
d) The connections of the diagonals at their ends should have sufficient over-strength and avoid a brittle mode of fracture. To this end requirements (h) and (i), presented above in the general rules, are also to be satisfied.
e) The beams and columns of the bracing system should have sufficient over-strength against the dissipative member (diagonal). It is considered that this requirement is fulfilled when:

$$N_{Rd}(M_{Ed}) \geq N_{Ed,G} + 1.10 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \tag{6.2}$$

where: $N_{Rd}(M_{Ed})$ the design resistance of the beam or column considering the interaction with the bending moment M_{Ed} produced by the seismic combination of actions, $N_{Ed,G}$, the axial force due to the non-seismic loads included in the seismic combination, γ_{ov} is the over-strength factor already commended (recommended value 1.25) and Ω is the ratio N_{Rd}/N_{Ed}, where N_{Rd} is the resistance of the diagonal (in tension) and $N_{Ed,E}$ the action due to the seismic design situation.
f) In V-bracings the beams should be designed to resist the non-seismic actions without consideration of the intermediate support provided by the diagonals.

g) In V-bracings the beams should also resist the unbalanced vertical seismic action applied to the beam after buckling of the compressive diagonal. The effect is calculated using $N_{pl,Rd}$ for the brace in tension and $\gamma_{pb} N_{pl,Rd}$ for the brace in compression. For γ_{pb} the value 0.30 is recommended, however different values could be adopted by the different National Annexes.

6.4.4.4 Specific rules for eccentric bracings to Eurocode 8

The following specific rules for eccentric bracings are intended to ensure that bending or shear plastic hinges would form in the links prior to any yielding or failure elsewhere in the bracing:

a) The web of the links should have a uniform thickness without strengthening plates and without holes or penetrations.
b) For links having I-sections, the following nominal values for the bending and shear resistances are considered:

$$M_{p,link} = f_y \cdot b \cdot t_f \cdot (d - t_f) \tag{6.3}$$

$$V_{p,link} = \left(\frac{f_y}{\sqrt{3}} \right) \cdot t_w \cdot (d - t_f) \tag{6.4}$$

where; b is the flange width, d the overall depth of the cross-section, t_f and t_w the thicknesses of the flange and the web respectively, and f_y the nominal value of the material yield stress.
c) Seismic links are classified into three categories following the type of the plastic mechanism:
   - long links, in which energy is dissipated mainly by bending plastic hinges,
   - short links, in which energy is dissipated mainly by yielding in shear, and
   - intermediate links where plastic mechanism involves simultaneously bending and shear.
d) In bracings' lay-outs, in which equal moments would form simultaneously at both ends of the link (Fig. 6.47h), links may be classified according to their length e. For links with I-sections the classification is:
   - long links

$$e > e_L = 3.0 \cdot \frac{M_{p,link}}{V_{p,link}} \tag{6.5}$$

   - short links

$$e > e_S = 1.6 \cdot \frac{M_{p,link}}{V_{p,link}} \tag{6.6}$$

   - intermediate links for e values between e_L and $e_{S.}$
e) A similar classification is given in EN 1998 for links in which only one plastic hinge would be formed at one end of the link (Fig. 6.47 i, j).
f) Specific formulae are also included in EN 1998 for the cross-section verification of the link.

g) When plastification takes place, the rotation θ of the link in relation to the adjacent members of the bracing, should be consistent with global deformations. To this end for long links, θ is recommended to be less than 0.02 rad, for short links less than 0.08 rad while for intermediate links a linear interpolation between the above values should be used.
h) Full depth web stiffeners should be provided on both sides of the link's web at the diagonals ends. These stiffeners should have a thickness not less than the greater value between $0.75t_w$ and 10 mm.
i) Detailed recommendations are included in the Code, depending on the type of the link, concerning additional required web stiffeners as well as the stiffeners welding on the web and the flange of the link cross-section.
j) Lateral supports should be provided, at the top and bottom flanges of the link, at the link ends, having sufficient axial resistance to ensure the formation of the plastic hinges. As an application rule, this resistance should be at least equal to 6% of the nominal plastic axial capacity of the flange.
k) For all other members of the bracing, apart the links, as well as for the connections of the link with the adjacent members, where an over-strength is required, similar formulae to (6.2) are given in EN 1998 to be respected.

6.4.5 Stressed skin design

Metal sheeting used for roofing and wall cladding in the buildings, acts, to some extent, as a diaphragm, increasing the stiffness of the structure and, consequently, limiting deformations and the effects of the actions. In usual practice this effect is not taken into account in structural analysis, as the cladding is not considered as a permanent structural element and as, in addition, the type of sheets and the fixing conditions are not always known in advance during the design phase. In addition the consideration of this effect leads only to saving of limited quantities of steel in the bracing systems, which are, in any case, necessary during erection while the effect of the holes elongation around cladding fixings is not sufficiently investigated.

However the sheeting diaphragm action could be included in structural design and the sheets could be considered as structural elements under some necessary conditions. In this case roofs could be treated as deep plate girders resisting transverse in-plane loads which they transfer to end gables or to intermediate frames. The sheeting panel would be considered as a web, resisting in plane loads in shear and the end members as flanges resisting axial tension and compression forces. In the same way wall panels could be treated as bracing systems acting as shear diaphragms to resist in plane actions. Such design is called "stressed skin design".

Reference to the subject is included in EN 1993-1-3 [6.11], where in a specific clause: "Stressed skin design" the conditions under which this design could be performed are indicated:

a) The use of the sheeting, beyond his primary purpose, is limited to act as shear diaphragm.
b) The diaphragm ends in longitudinal edge members which act as flanges and receive forces coming from the diaphragm action.

c) Suitable and sufficient structural connections are designed to transmit diaphragm forces to the main structural steel members as well as to the edge members acting as flanges.
d) The sheeting is treated as a permanent structural component which cannot be removed without appropriate consideration. This approach, to include a stressed skin design in the analysis and the calculations, is necessary to be indicated in the project specification and the drawings.

Some additional recommendations included in EN 1993-1-3 are the following:

a) Stressed skin design may be used predominantly in low rise buildings.
b) Sheets, used additionally to their main purpose as diaphragm elements, may not be selected to resist permanent external loads, as, for instance, loads coming from plants.
c) The seams between adjacent sheets should be fastened by rivets, self-drilling screws, welds or other type of fastening with a spacing not more than 500 mm.
d) Small openings, up to 3% of the relevant area, may be arranged without a special calculation, provided that the total number of fasteners is not reduced. Openings up to 15% of the relevant area could be arranged after justification by calculations. In cases of openings covering more than 15%, the diaphragm area should be split into smaller areas.
e) To ensure that any sheeting deterioration will be visible during its main function in bending, before stressed skin action is developed, the shear stress due to the above action should not exceed the 25% of the yield stress design value.

6.5 Secondary structural elements

6.5.1 The purlins

The purlins, as already mentioned in 6.1, are beams bridging the span between main structural elements (typical frames or trusses). They support cladding and carry loads applied to the roof transferring them to the main frames. Some purlins may also participate as members of the horizontal bracing systems (see 6.4.2). For purlins hot rolled I-sections with a depth usually between 100 and 180 mm or, more often, cold-formed members of a cross-section depth between 140 and 300 mm, are used. For the spacing between purlins reference is also made in 6.1. At the end areas of the roofs, where wind pressures or uplift have greater values compared to the ones at the rest of the roof, or larger values of the snow have, sometimes, to be considered (due to probable adjacent buildings, parapets etc.), it might be decided to adopt locally stronger cross-sections or smaller spacing between purlins. In cases of greater distances between main frames, for instance 10 to 12 m, truss form prefabricated beams (joists) could also be used as purlins (Fig. 6.51).

In Fig. 6.52 typical cross-sections of cold-formed purlins are presented. The above purlins are formed using 1.5 to 3 mm thickness galvanized steel sheets. The most commonly used cross-sections are the C and Z types. In Z-types the flanges are,

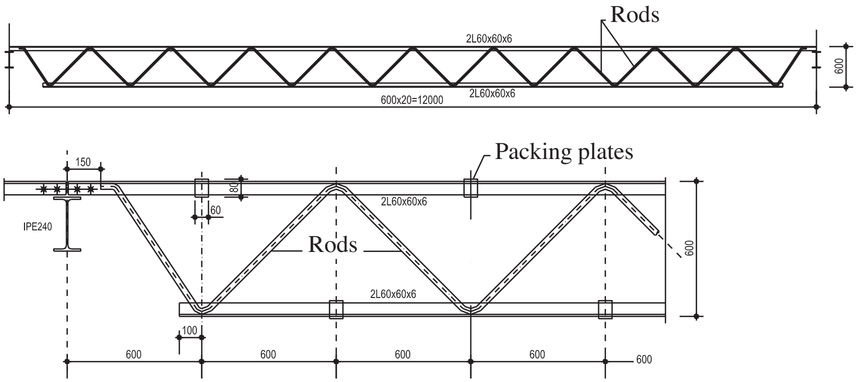


Fig. 6.51. Joists as long span purlins

usually, of unequal width to permit an overlapping between successive elements. Various cross-sections are developed and tested by manufacturers who give, in design Tables, the capacity of the purlin against vertical loading with respect to the span, the distances between purlins and the other relevant conditions (continuity, laterally supporting elements, anti-sag bars etc.).

Purlins could be introduced as simply supported beams, as continuous beams, with or without intermediate articulations, with total or partial continuity. In the case of articulations, their location influences the bending moment distribution of the continuous purlins. In cases of larger purlin spans inclined bars could be used to support the purlins. These bars offer in addition lateral support to the lower flange of the rafter or to the lower chord of the truss (Fig. 6.53). In cold-formed Z sections adjacent elements are usually connected semi-continuously by overlapping, using bolts in the web of the sections (Fig. 6.54a). Semi-continuity can also be achieved by sleeves placed over the supports (Fig. 6.54b).

Cladding metal sheets offer to the purlins full or partial lateral restraint against lateral buckling, in cases they have sufficient rigidity and are rigidly fixed to the purlins, depending on the type, the number and the strength of the fixings. Lateral-torsional buckling has to be examined for both cases of permanent plus live vertical loads and under uplift conditions. Information and application methods to determine the corresponding resistances are included in EN 1993-1-3: "Supplementary rules for cold-formed thin gauge members and sheeting", Chapter 10: Beams restraint by

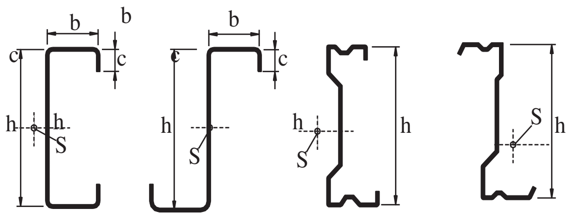


Fig. 6.52. Alternative cold-formed cross-sections for purlins. Shear centres position

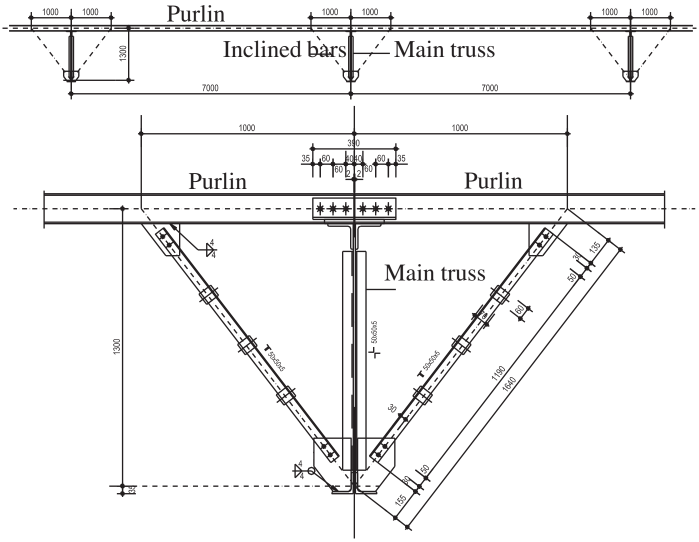


Fig. 6.53. Inclined bars for additional support to the purlins and lateral support of the truss lower chord

sheeting". When the sheets are thin, with limited rigidity and weak fixing conditions, it is on the safe side to consider that there is no lateral protection. In all cases the detailing at the supports of the purlins to the rafters should be sufficiently rigid, to avoid twisting in order to be compatible with the calculation assumptions (Fig. 6.55). The purlins should be oriented in such a way to decrease the distance between the

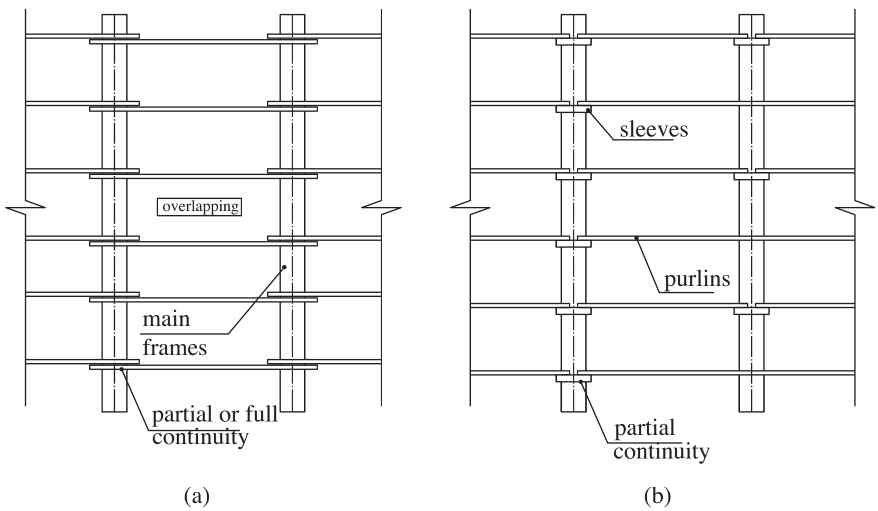


Fig. 6.54. Alternative arrangements for Z cold-formed purlins

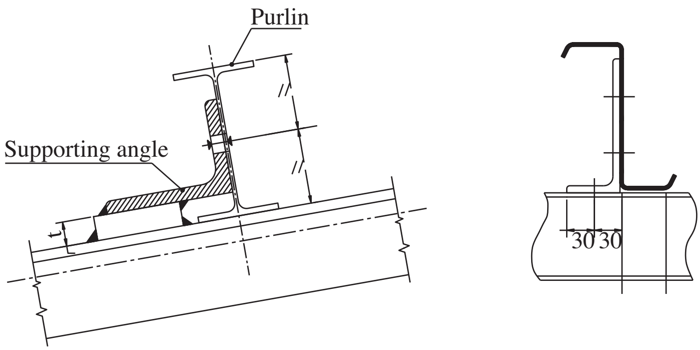
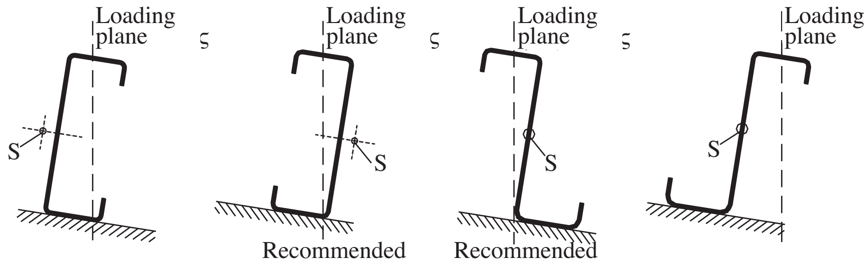


Fig. 6.55. Support details of purlins

shear center of the cross-section and the vertical loading and therefore to reduce torsional phenomena (Fig. 6.56).



(a) (b) (c) (d)

Fig. 6.56. Cold-formed purlins positions. Distances between loading plane and shear center

Purlins are subjected, apart from bending around their strong principal axis, to a secondary bending around their weak axis, due to the slope of the roof. To limit this secondary bending and the related deformations, especially in cases of significant slopes, anti-sag bars are provided joining adjacent purlins and creating an intermediate lateral support (Fig. 6.57a). Anti-sag bars are usually placed at the 1/2 or 1/3 of the span and are anchored to the top of the roof (Fig. 6.57a, b). Circular compact cross-sections, of 12 to 16 mm diameter, are commonly used for anti-sag bars, bolted at both ends. For the lateral support of the purlins at the top of the roof, a tube able to resist compression forces could be used (Fig. 6.57b). Each bar transfers the tensile forces coming up from all purlins of a lower level. Anti-sag bars are, finally, also used for the alignment of the purlins, if needed.

Concerning the serviceability limit state the deformations of the purlins should be appropriately limited. According to the provisions of EN 1990 [6.8] for non-accessible roofs a limitation of the 1/250 of the span has to be respected for the characteristic SLS combination of loading and, additionally, of the 1/200 of the span due to the variable actions only.

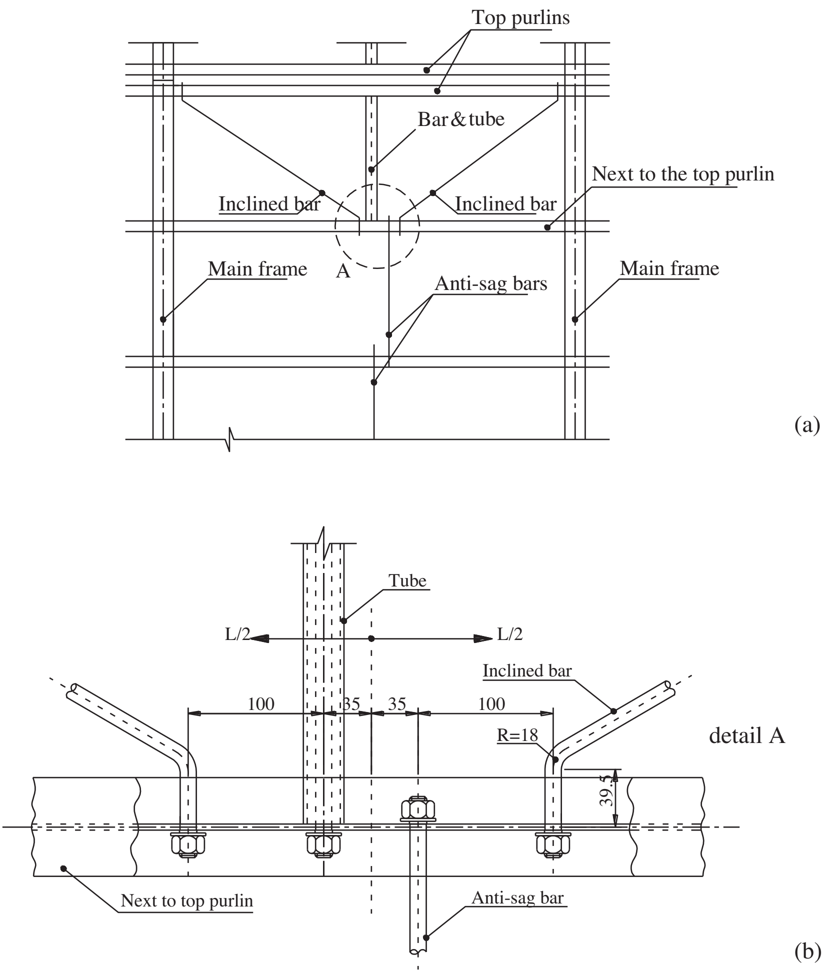


Fig. 6.57. Anti-sag bars. Anchoring at the top of the roof

6.5.2 The side rails

Side rails are horizontal beams on the external walls of the building, supported by the columns. They support the cladding panels and carry mainly wind forces. Side rails are subjected to similar, as the purlins, conditions, and they should be treated similar to them as far as the lateral support provided by the sheeting and the direction of the wind loading (pressure or suction) are concerned. Reference is already done in 6.1.

Side rails are usually of the same types of cross-sections as the purlins. Their spacing is between 1.50 and 2.50 m. They are usually formed as simply supported beams between columns but could also follow other structural systems similar to the purlins. Anti-sag bars are also used at 1/2 or 1/3 of their span to limit lateral bending and to help the alignment. One side rail is in all cases placed at the upper and the

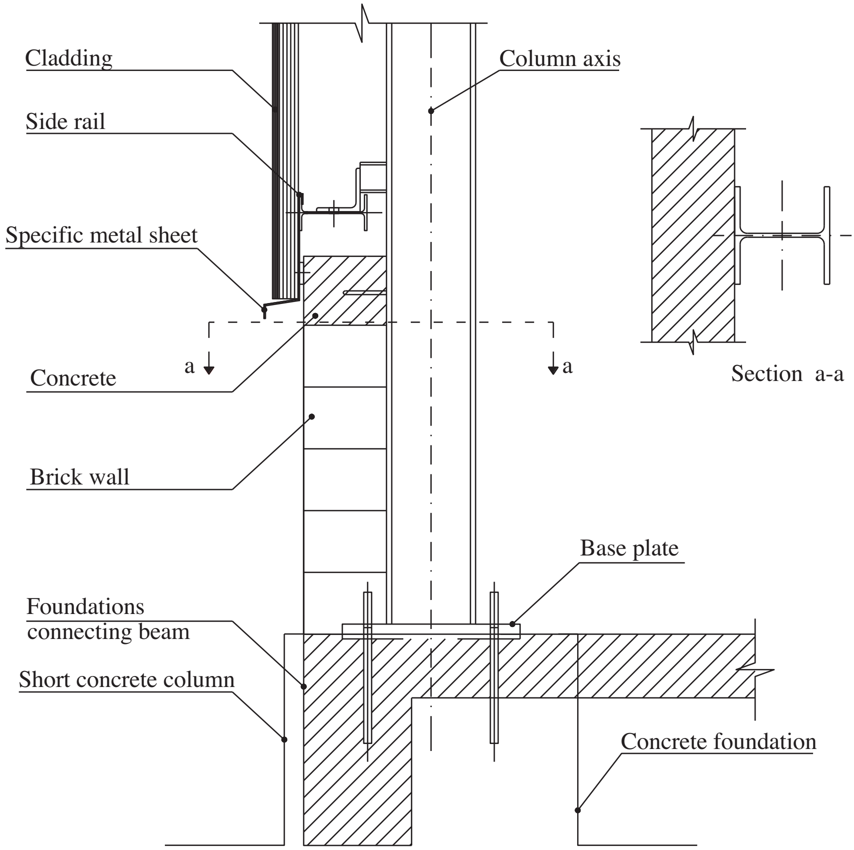


Fig. 6.58. Lower purlin arrangement

lower level of doors and windows as well as at the lowest level of the lateral sheeting (Fig. 6.58).

An alternative to side rails and lateral sheeting are liner trays (Fig. 6.59). They have the shape of a large channel-type section with a wide flange, two webs and two narrow flanges which should be laterally restrained by the attached profiled steel sheeting. In the interior thermal insulation material is incorporated. Liner trays have usually a depth h of 60 to 200 mm, a width b_u of 300 to 600 mm and sheet thickness of 1.5 to 2.5 mm. Their orientation may be horizontal or vertical between

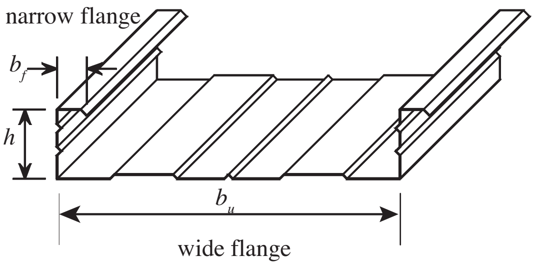


Fig. 6.59. Liner trays

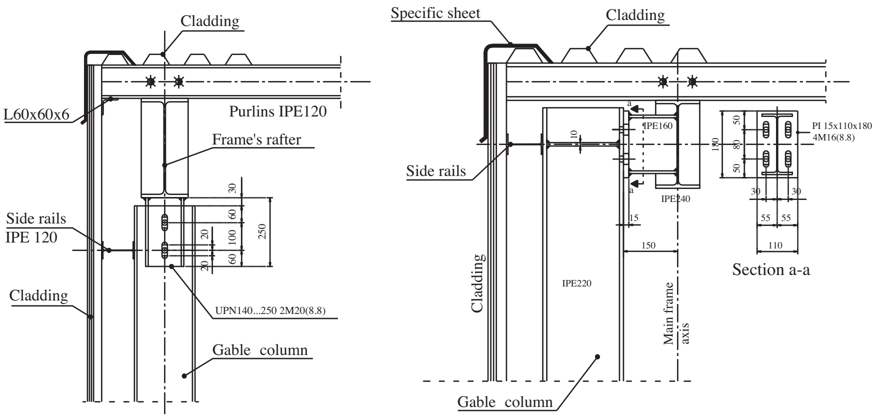
structural horizontal members. Methods to determine the resistance of the liner trays are included in EN 1993-1-3.

6.5.3 Gable wall columns

As already mentioned in 6.1, in the gable walls of the building additional columns are needed to support the side rails. These columns could be arranged without influencing the behavior of the end frames or participate in a gable frame different from the typical one. In the first case gable columns are formed as pinned members at both ends provided with elongated holes at the top to absorb frame deflections, without transferring compression forces to the columns (Fig. 6.60, 6.61).

Gable columns are usually positioned below nodes of the horizontal bracing. The selected positions are also related to the need of openings (usually doors) provided by the architectural design. Gable columns supported as pinned members, are structural elements subjected mainly to bending due to wind actions. They transfer wind pressures to the horizontal bracing system on the top and to their foundation on the bottom. In relatively tall buildings a fixed end at the base of the column decreases the transverse force applied to the bracing at the top.

Gable columns may be placed either on the axis of the end frame (Fig. 6.60a) or out of this axis (Fig. 6.60b). For gable frames having a truss as horizontal member, the gable columns could be placed eccentric to the frame axis, arriving up to the top chord of the truss and transferring directly the wind forces to the horizontal bracing system. Alternatively they may be placed concentric to this axis (Fig. 6.61) in which case wind forces are transferred to the level of the aforementioned horizontal bracing system by a local vertical bracing or by an additional secondary horizontal bracing at the level of the lower chord.



(a) (b)

Fig. 6.60. Gable column connected at the gable frame using elongated holes (a) in the axis of the frame (b) out of the axis

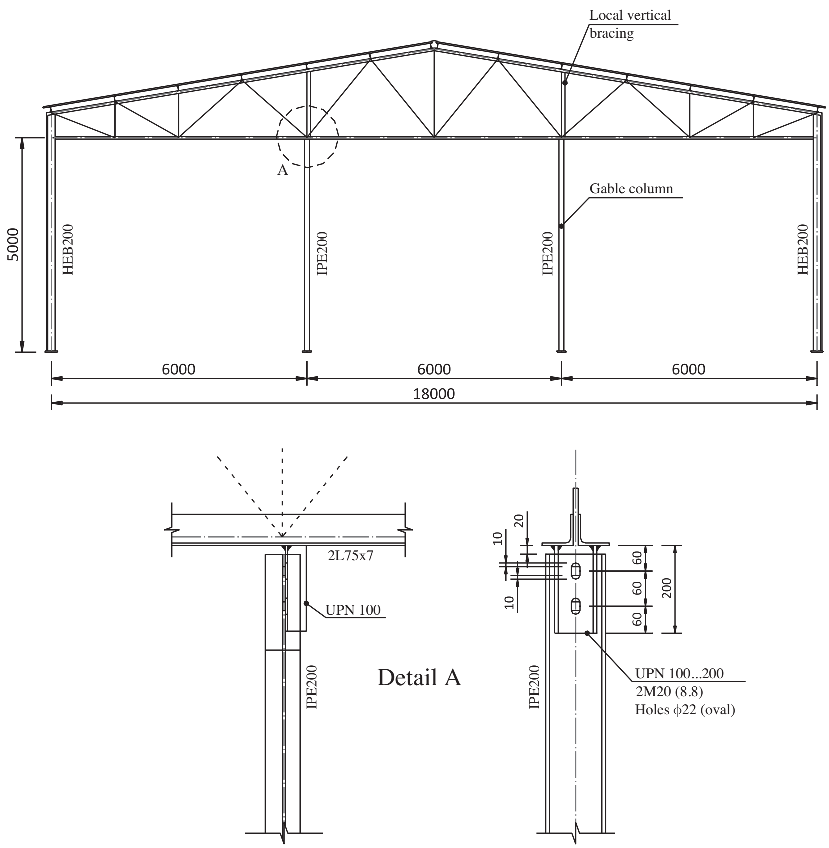


Fig. 6.61. Gable columns placed in the axis of an end truss

Figure 6.62 shows a front view of a steel structure including the end frame, the gable columns, the side rails and the associated anti-sag bars. For large buildings with sliding doors the corresponding details should be formed in relation to the weight, dimensions and type of the doors. Light sliding doors are suspended from horizontal beams (Fig. 6.63). Heavier doors usually slide on rails and are simply laterally

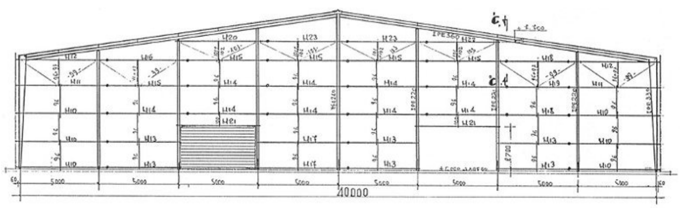


Fig. 6.62. Typical front view of the structural elements

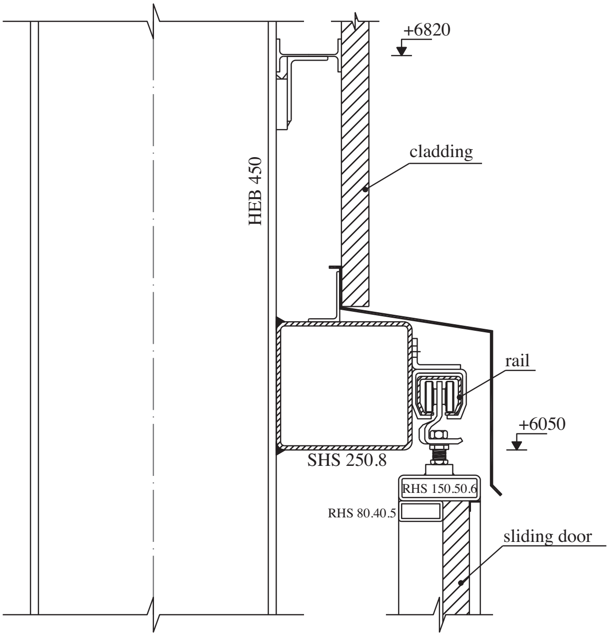


Fig. 6.63. Light sliding door suspended by SHS beam

supported at their top. Related details are shown in Fig. 6.64 for a shipyard and Fig. 6.65 for an airplane hangar.

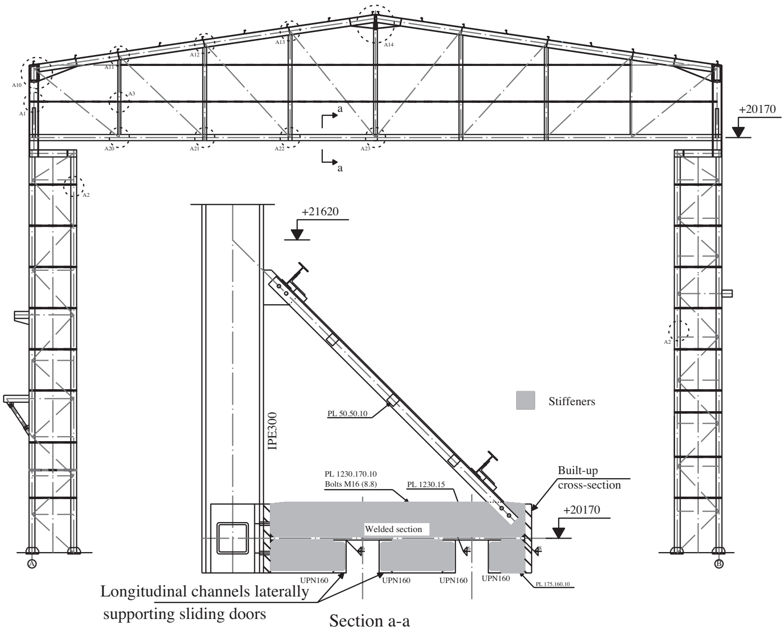


Fig. 6.64. Supplementary structures on an end frame to support laterally sliding doors

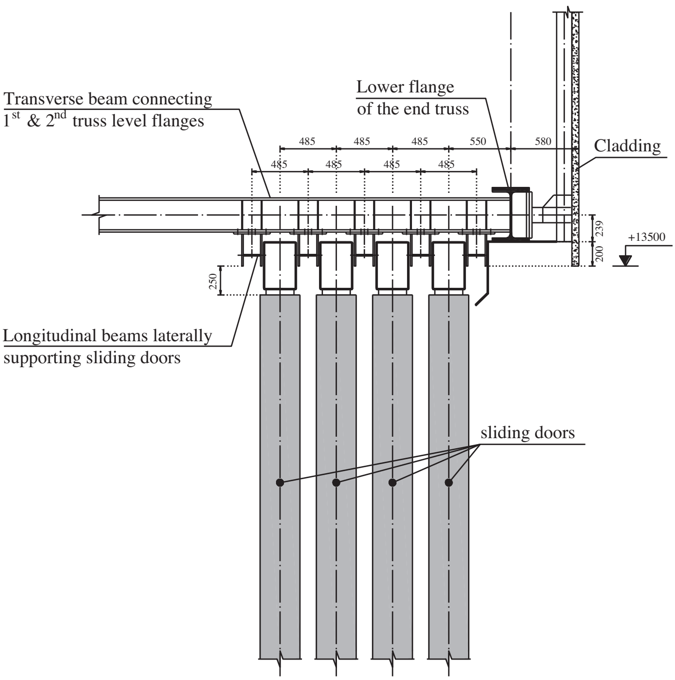


Fig. 6.65. Auxiliary beams to support laterally sliding doors

6.6 Crane supporting beams

6.6.1 Introduction

In the majority of industrial buildings and warehouses, cranes serve to remove and handle products during their fabrication or ready to be used or consumed. Cranes are supported by beams running usually along the sides of the building. Crane supporting beams are specific structural members in the sense that, in addition to strength requirements, they must satisfy demanding serviceability and fatigue criteria.

A typical arrangement of a crane, in the interior of a building, is shown in Fig. 6.66. The crane moves on wheels along the crane runway supporting beams and spans their distance forming a crane bridge. The main structural elements of the crane bridge are two beams with box cross-sections ending at the lateral members to which the wheels are connected. The wheels, two at each end for the usual cases or four for higher hoist loads, move on rails fastened to the top of the crane supporting beams. The crab moves transversely on rails placed on the top of the crane bridge and incorporates a hoist, ending at its lower level at a hook used for attaching hoisted material. Cranes moving along the top flange of the supporting beams, which is the usual case, are called top mounted cranes.

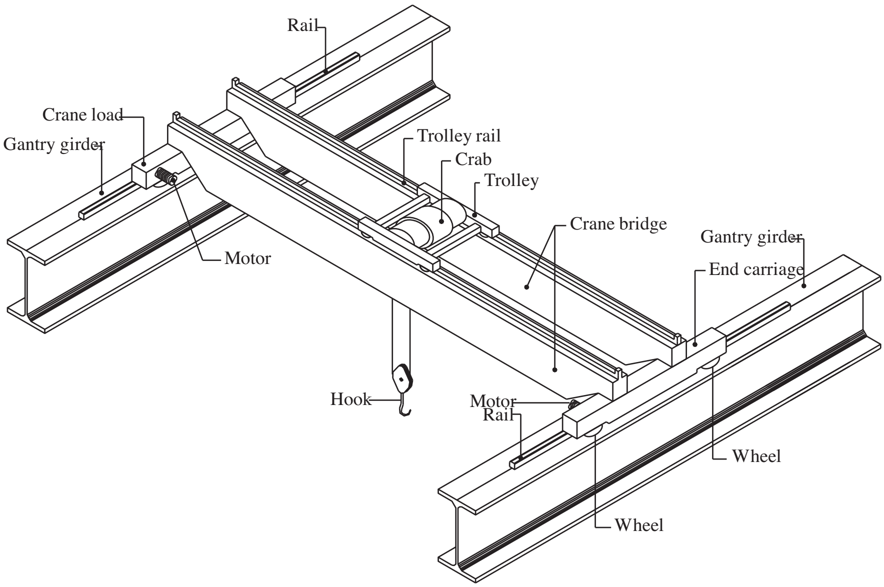


Fig. 6.66. Typical arrangement of a crane

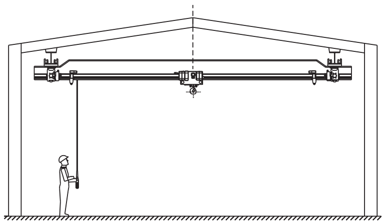


Fig. 6.67. Underslung crane

In some cases, instead of hooks, the hoist block is equipped with buckets or grabs, depending on the nature of the material to be moved. In shipyards or steel building construction industries, where plane steel elements of frequently significant dimensions should be removed, instead of hooks, a magnet operation is provided. The crane bridge can incorporate one or more hoists. In cases with smaller hoisting capacities, the crane can move below the runway beams and be supported on their bottom flanges which is the case of the underslung cranes (Fig. 6.67). For both top mounted or underslung cranes, called overhead travelling cranes, the hook can practically reach any location of the building. In monorails, where one runway beam exists, on the lower flange of which the hoist is supported, the hook can have in plan a linear movement only (Fig. 6.68).

Alternative cross-sectional shapes for the rails which are fixed on the top of the runway beams are shown in Fig. 6.69. For smaller hoisting loads a simple rectangular cross-section is usually used, while for higher hoisting capacities a shape of train rail is selected. The rails are connected to the beams according to a “rigid” or “independent” way (see 6.6.7).

For the analysis and the design of the crane supporting beams according to the Eurocodes two parts apply: (a) Eurocode 1, Part 3: “Actions on structures. Actions induced by cranes and machinery” (EN 1991-3), [6.12] and (b) Eurocode 3, Part 6:

"Design of steel structures. Crane supporting structures" (EN 1993-6), [6.13]. This section 6.6 presents the main provisions and recommendations of the aforementioned codes. Provisions for the cranes themselves are provided by EN 13001 [6.14, 6.15].

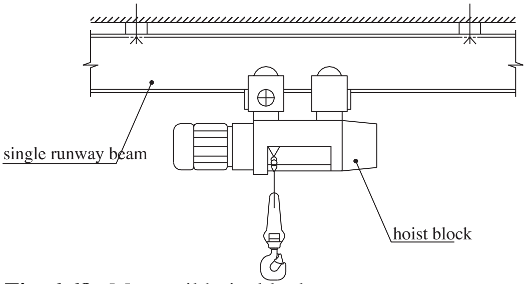


Fig. 6.68. Monorail hoist block

Cranes are industrial products and are therefore not, in general, part of the structural design. The crane suppliers ensure strength, serviceability and the electromechanical characteristics of the crane. In addition they provide the clients with information on the geometrical data of the crane bridge (minimum distance between hook and axis of the runway beam, distance between the top level of the hook and the top of the rail, distance required between top of the rail and the roof, distance between wheels etc. see Fig. 6.70).

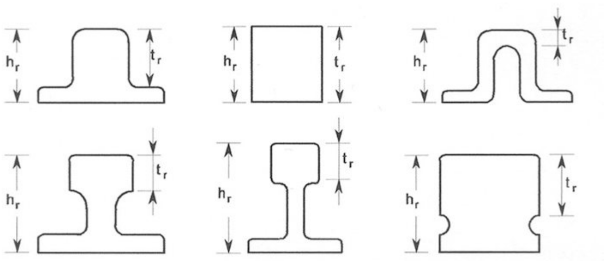


Fig. 6.69. Alternative cross-sections for crane rails (EN 1993-6)

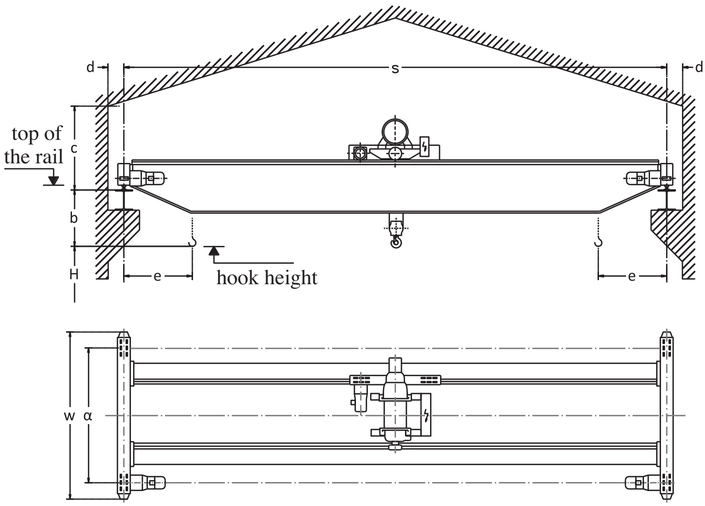


Fig. 6.70. Geometrical data of a crane bridge

Starting from the top level of the hook, which is an operational requirement of the building, and based on the above data, the minimum clear height of the hall may be determined. The crane supplier provides also the maximum and minimum reactions of the crane per wheel (actions on the building).

The design working life of a crane supporting structure is the period during which the structure can be fully functional. EN 1993-6 recommends a life time of 25 years. Nevertheless for beams non-intensively used a life time of 50 years could be appropriate. Structural components of the crane supporting structure, like crane rails and rail fixings, which cannot reach, with adequate reliability, the total working life time should be replaceable.

6.6.2 Actions induced by crane bridges on the runway beams

6.6.2.1 General

The crane bridge induces to the runway beams vertical and horizontal loads. The vertical loads correspond to the self-weight of the crane (including the hoisting equipment) and to the hoisted load. Horizontal forces develop during the crane acceleration, when it starts to move, or deceleration at the end of the movement. Horizontal forces also develop during the movement of the crane, with a constant speed, due to the possibility of skewing. It is evident that the above forces apply to the runway beams as concentrated loads through the wheels. Horizontal dynamic actions, lateral or longitudinal, due to the crane operation on the runway beams are called "crane surge". The connectors transmitting the crane surge from the beam to the supports are the surge connectors.

Cranes can also produce horizontal actions due to an accidental collision with the buffers (buffer forces), placed at the ends of the runway beams or a collision with obstacles (tilting forces). The above accidental forces have, in general, a local effect.

As already mentioned, actions induced by cranes are the subject of EN 1991-3. In this code the simultaneous application of the various main and accidental actions is taken into account by considering groups of loads. The dynamic character of these actions is also considered by introducing dynamic magnification factors to obtain equivalent static forces. The values of these factors depend on the type of the lifting equipment, the type of the crane and the lifting speed.

After completion of the erection, tests are usually performed for the quality control and the integrity verification of the crane. To this end an appropriate procedure is to be specified and the crane supporting beams should also to be designed for forces introduced during this procedure (test loading). Finally an analytic procedure must also be followed for the assessment of the fatigue effect on the runway beams and the associated connections.

6.6.2.2 Vertical loads

Vertical loads are due to the self-weight of the crane, the weight of the lifting equipment (movable elements including mechanical and electrical parts) and the hoisted load. They apply to the crane supporting beams as concentrated loads through the wheels. As the lifting equipment and the hoisted load can move transversely to the crane's direction of movement, the loads applied to the two supporting runway beams could be different, the maximum value on the one beam (uploading at the nearest to the one beam position) coexist with the minimum value on the other.

To consider the dynamic effect of the applied loads the statically calculated forces must be increased by using dynamic magnification factors to obtain equivalent static values. Crane suppliers provide, for different capacities and spans, the aforementioned max and min reactions. These nominal values should be taken as characteristic values of the vertical loads. If sufficient information is not available during structural design, EN 1991-3 provides values for the dynamic factors which are indicated in Table 6.1. In this Table ϕ_1 corresponds to the excitation of the crane structure due to lifting of the hoist load from the ground and it is applied to the self-weight of the crane. Factor ϕ_2 applies to the hoist load, in order to consider the dynamic effects of its transfer from the ground to the crane. In EN 1991-3 the cranes are classified in four hoisting classes (HC1, HC2, HC3, HC4) depending on the particular type of the crane and the corresponding intensity of the dynamic effect. The classification for the different types of cranes is included in Table 6.11 in section 6.6.5 and is related to the value of ϕ_2. In case of a possible sudden release of the payload, as it is, for instance, the case of cranes with grabs or magnets, factor ϕ_3 has to be applied. Factor ϕ_4 is used in specific cases indicated in Table 6.1.

During erection the verticality of the columns is ensured within the acceptable limits of erection imperfections, therefore the runway beam can deviate, left or right side, from the ideal rectilinear geometry to a slightly crooked line. Since the rails, which are fixed on the beams after completion of the erection, must follow a strictly straight line, there are, along the beams, eccentricities between the rail (and the applied forces on the rails) and the center of the beams cross-section. The influence of this eccentricity on the capacity of the beams must be taken into account in the analysis and the design. As an application rule EN 1993-6 recommends, for this eccentricity, a value equal to 1/4 of the rail width (see Fig. 6.71a).

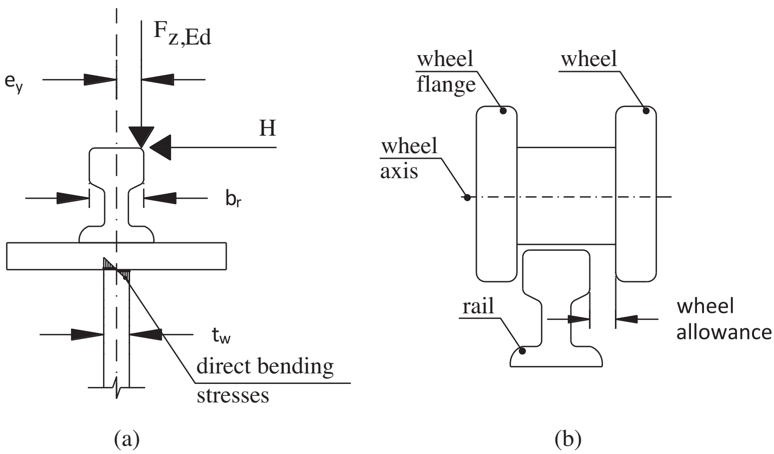


Fig. 6.71. Eccentric action of the vertical loads. Eccentricity $e_y = 1/4b_r$. Bending stresses developed on the web

Table 6.1. Dynamic factors ϕ_i for vertical loads (EN 1991-3)

	Values of dynamic factors		
ϕ_1	$0.9 < \phi_1 < 1.1$ The two values 1.1 and 0.9 reflect the upper and lower values of the vibrational pulses.		
ϕ_2	$\phi_2 = \phi_{2,min} + \beta_2 v_h$ v_h – steady hoisting speed in m/s $\phi_{2,min}$ and β_2 depending on the hoisting class as below		
	Hoisting class of appliance	β_2	$\phi_{2,\mathrm{min}}$
	HC1	0.17	1.05
	HC2	0.34	1.10
	HC3	0.51	1.15
	HC4	0.68	1.20
ϕ_3	$\phi_3 = 1 - \frac{\Delta m}{m}(1+\beta_3)$ where Δm released or dropped part of the hoisting mass m total hoisting mass $\beta_3 = 0.5$ for cranes equipped with grabs or similar slow- release devices $\beta_3 = 1.0$ for cranes equipped with magnets or similar rapid-release devices		
ϕ_4	$\phi_4 = 1.0$ provided that the tolerances for rail tracks as specified in EN 1993-6 are observed.		

NOTE: If the tolerances for rail tracks as specified in EN 1993-6 are not observed, the dynamic factor ϕ_4 can be determined with the model provided by EN 13001-2.

6.6.2.3 Horizontal loads during acceleration or deceleration of the crane

When the crane accelerates or decelerates the drive force K (see Figure included in Table 6.2) applies, through the drive wheels, in the axis of movement while the mass center S of the system (crane and hoisted load) is located, in general, at a distance from this axis. The moment M resulting from this eccentricity is counterbalanced by couples of forces H_T developing between flanged wheels of the same rail. At the same time longitudinal forces also develop at the contact surface between rails and driven wheels.

When the hoisted load is in the closest to a rail position, M and therefore H_T take their maximum values which should be used in calculations. Forces H_T apply dynamically, therefore amplification factors should be used in order to determine their design values. If the dynamic effect is not included in the specification documents of the crane supplier the values indicated in the regulations are to be used. In Table 6.2 the formulae included in EN 1991-3 are presented including values for the dynamic factor ϕ_5. The drive force K in a driven wheel should be taken such that wheel spinning is prevented and should be obtained from the crane supplier. Where there is no a wheel controlled system in use, the related provisions of EN 1991-3 for K should also be taken from Table 6.2. In these forces only the effects of a small unavoidable misalignment between the hoist load and the crab is included. Large values of this misalignment are, in any case, not allowed.

Table 6.2. Horizontal forces due to the acceleration of the crane (EN 1991-3)

$M = Kl_S \qquad l_S = (\xi_1 - 0.5)l,$

$H_{T,1} = \phi_5 \xi_2 \dfrac{M}{\alpha}, \qquad H_{T,2} = \phi_5 \xi_1 \dfrac{M}{\alpha}, \qquad H_{L,2} = H_{L,2} = \phi_5 K/2$

$K = \mu m_v Q_{r,\min}$ (for a single wheel drive)

where

μ the friction factor equal to 0.20 for a steel-steel contact and 0.50 for a steel-rubber contact (recommended values)

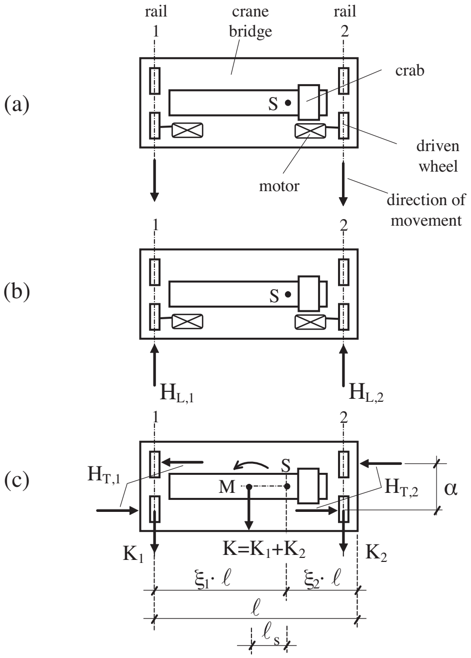
m_v the number of single wheel drives

$Q_{r,\min}$ the minimum load per wheel of the unloaded crane

NOTE: For a central wheel drive, which is not the case for modern cranes, a different value for K has to be used (see EN 1991-3).

Values for ϕ_5:

$\phi_5 = 1.0$	for centrifugal forces
$1.0 \leqslant \phi_5 \leqslant 1.5$	for systems where forces change smoothly
$1.5 \leqslant \phi_5 \leqslant 2.0$	for cases where sudden changes can occur
$\phi_5 = 3.0$	for drives with considerable backlash



6.6.2.4 Horizontal forces due to skewing of the crane

Due to the small necessary allowance between wheel flanges and the rail (Fig. 6.71b), the crane may be placed between the two parallel rails in a slightly oblique way (skewing), rotating about an instantaneous rotation center and applying to the rails, while travelling in a steady state motion, longitudinal and transverse horizontal forces (see Fig. 6.72).

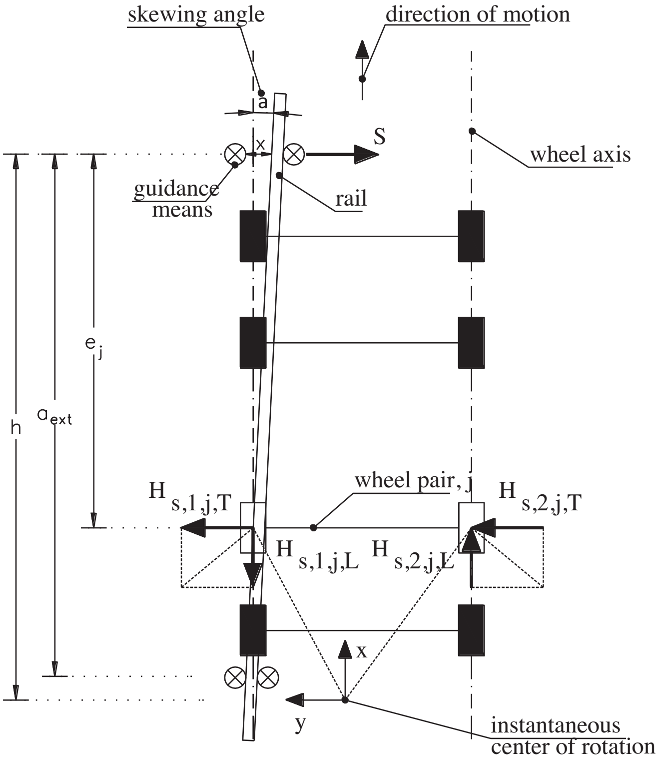


Fig. 6.72. Skewing forces

The crane is brought back through guidance means (usually rollers) which are placed close to some selected wheels, slightly forwarding or following these wheels, depending on the system adopted. The crane can also be guided by means of wheel flanges. The direction and the values of the skewing forces depend on the type and the arrangement of the guidance means and on the type of the wheel driving. Skewing forces develop during travelling of the crane with a constant speed. Therefore are not combined with the horizontal forces that appear, as above, during the acceleration of the crane and they haven't a dynamic effect. Their characteristic values are usually given by the crane supplier. As an alternative they can be calculated following a code procedure.

Following the EN 1991-3 procedure, skewing forces could be determined using the following relations (6.7) and (6.8) for the lateral force corresponding to the guidance means and to the wheels

$$S = f \cdot \lambda_{S,j} \cdot \sum Q_r \qquad (6.7)$$

$$H_{S,i,j,L\,or\,T} = f \cdot \lambda_{S,i,j,L\,or\,T} \cdot \sum Q_r \qquad (6.8)$$

In the above formulae index S denotes skewing. Index i corresponds to the number of the rail (1 or 2) while index j corresponds to the pair of wheels (1 for the front wheels). $\sum Q_r$ is the sum of all wheel vertical loads of the loaded crane. Index L represents a longitudinal action while T a transverse one. Force S is applied to one of the two rails (Fig. 6.72). When the crane is guided by the wheel flanges, S is applied at the position of the wheel axis. Coefficient f is related to the skewing angle which can be approximately taken equal to 0.015 rad. In this case f is equal to 0.293. In EN 1991-3 detailed information is given to calculate, if needed, a more accurate smaller value for f considering constructional details of the guiding system (space between guidance means and rails etc.).

Values of the coefficients λ can be calculated using the information given in Table 6.3 through the distance h between the instantaneous center of rotation and the relevant guidance means. The value of h depends on the way the wheels are fixed on their axes. Two driven wheels of the same pair (front wheels or followers) can have a common axis (coupled wheels, letter C, central wheel drive) or, which is the most common case, each wheel of the pair has its own axis (independent wheels, letter I, single wheel drive). The axis could also be fixed on the wheel (letter F) or having the possibility to slightly move, perpendicular to the wheel (movable axis, letter M). Following the above, for each pair of wheels we can have one of the four alternative combinations, shown in Table 6.3. If the crane is guided by the wheel flanges without specific guidance means, h is the distance between the center of rotation and the axis of the front wheels.

6.6.2.5 Accidental loads

At the ends of the crane runs structural crane stops (buffers) are placed, intended to stop the crane or the hoist, reaching the end of the runway. The buffers are usually fixed on the runway beams. The forces applied on the crane supporting structures when a collision with the buffers happens should be considered in the calculations.

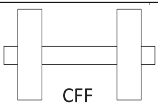
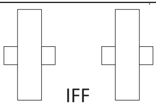
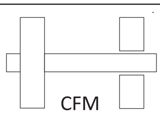
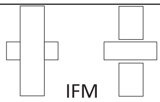
EN 1991-3 gives information for the calculation of the buffer force arising from the kinetic energy of all relevant parts of the crane. It is recommended to obtain this force (in N) from expression (6.9):

$$H_B = \phi_7 \cdot v_1 \cdot (m_c \cdot S_B)^{1/2} \qquad (6.9)$$

where v_1 is the 70% of the nominal travelling speed of the crane (m/sec), m_c is the mass of the crane and the hoist load (kg) corresponding to each buffer, S_B is the spring constant of the buffer (N/m) and ϕ_7 is the dynamic coefficient which conservatively could be taken equal to 1,60. EN 1991-3 and EN 13001-2 give additional

Table 6.3. Definition of $\lambda_{s,i,j,k}$ values (EN 1991-3)

System	$\lambda_{S,j}$	$\lambda_{S,1,j,L}$	$\lambda_{S,1,j,T}$	$\lambda_{S,2,j,L}$	$\lambda_{S,2,j,T}$
CFF	$1-\frac{\sum e_j}{nh}$	$\frac{\xi_1\xi_2}{n}\frac{l}{h}$	$\frac{\xi_2}{n}\left(1-\frac{e_j}{h}\right)$	$\frac{\xi_1\xi_2}{n}\frac{l}{h}$	$\frac{\xi_1}{n}\left(1-\frac{e_j}{h}\right)$
IFF		0	$\frac{\xi_2}{n}\left(1-\frac{e_j}{h}\right)$	0	$\frac{\xi_1}{n}\left(1-\frac{e_j}{h}\right)$
CFM	$\xi_2\left(1-\frac{\sum e_j}{nh}\right)$	$\frac{\xi_1\xi_2}{n}\frac{l}{h}$	$\frac{\xi_2}{n}\left(1-\frac{e_j}{h}\right)$	$\frac{\xi_1\xi_2}{n}\frac{l}{h}$	0
IFM		0	$\frac{\xi_2}{n}\left(1-\frac{e_j}{h}\right)$	0	0

Fixing of wheels according to lateral movements	Combination of wheel pairs		h
	coupled (c)	independent (i)	
Fixed/Fixed FF			$\frac{m\xi_1\xi_2 l^2+\sum e_j^2}{\sum e_j}$
Fixed/Moveable FM			$\frac{m\xi_1 l^2+\sum e_j^2}{\sum e_j}$

Where:
n is the number of wheel pairs;
$\xi_1 l$ is the distance of the instantaneous centre of rotation from rail 1;
$\xi_2 l$ is the distance of the instantaneous centre of rotation from rail 2;
l is the span of the appliance;
e_j is the distance of the wheel pair j from the relevant guidance means;
h is the distance between the instantaneous centre of rotation and the relevant guidance means;
m is the number of pairs of coupled wheels ($m = 0$ for independent wheel pairs).

information to determine, a more precise smaller value for ϕ_7 (between 1.25 and 1.60). Buffers could also be placed to ensure the safe movement of the crabs. In case that the payload is free to swing, the buffer force can be taken equal to the 10% of the sum of the hoist load and the weight of the crab. Otherwise the buffer force could be determined as previously explained for the cranes. Accidental loads are introduced in the corresponding load combinations with a partial safety factor γ=1.0.

In case that a crane with horizontally restrained loads tilts when its load or lifting equipment collides with an obstacle, the resulting static force (without a dynamic amplification factor) should be considered (tilting forces).

6.6.2.6 Test loading

When tests are performed after the erection of the building for the acceptance and the quality control of the crane and the crane supporting structures, the relevant structures must be checked, during design, against the test loading conditions.

According to EN 1991-3 the crane and its supporting structure should be subjected to two different tests: (a) a static test by loading the crane, without use of the drives, with a load equal at least to the 125% of the nominal hoist load. In the calculations no dynamic amplification factor is to be applied and (b) a dynamic test. In this case the crane is moved by the drives, hoisting a load at least equal to the 110% of the nominal value. In the calculations a dynamic factor of

$$\phi_6 = 0.5 \cdot (1 + \phi_2) \tag{6.10}$$

is recommended to be used. To the above testing loads, considered as actions, a partial safety factor $\gamma = 1.10$ should be applied in the calculations.

6.6.2.7 Load combinations

The crane action is introduced in the analysis as variable action to be combined with the permanent and other variable, accidental and seismic actions following the codes used and applying partial safety and combination coefficients recommended by these codes.

Following EN 1991-3 vertical and horizontal forces induced by cranes to the overall structure are combined according to Table 6.4 with the indicated corresponding dynamic factors. Each column of the Table should be considered as a group of loads defining one single characteristic, variable, crane action which will be combined with the non-crane loads. In this way there are ten alternative crane actions to consider. For a permanent crane action with an unfavorable effect a partial safety factor of $\gamma = 1.35$ is recommended by EN 1991-3 (for a favorable effect $\gamma = 1.00$). For the variable crane actions, the value $\gamma = 1.35$ is also recommended while for all other variable actions (live load, wind, snow) $\gamma = 1.50$ is applicable. As already mentioned, accidental crane actions are to be introduced in the accidental combinations, without a safety amplification ($\gamma = 1.0$). In the load combinations, where the crane action is the main variable action, the combination coefficients ψ are recommended to take the values $\psi_0 = 1.00$ for the usual combinations and $\psi_1 = 0.90$ for the accidental and frequent combinations. Finally for the seismic combination it is recommended to consider a value of ψ_2 equal to the ratio between the permanent and the total crane action. It is also noted that for γ and ψ coefficients, different values from the above could be adopted by the National Annexes of the countries applying Eurocodes.

In industrial and similar buildings more than one cranes could operate simultaneously along the same rails or along adjacent halls. EN 1991-3 indicates (Table 6.5) the number of cranes which will be considered in the calculations as acting together. Cranes that are required to cooperate in hoisting the same load shall be treated as a single crane.

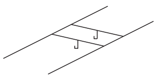
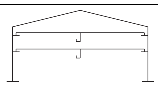
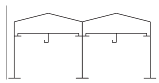
It is evident, even in the case of one crane in a building, that the alternative positions and crane actions are numerous (many possible positions along the hall, hoist load close to the one or the other rail, horizontal forces directed to the left or to the right side) leading to a significant number of alternative crane actions. It is also clear that for each structural member a different position of the crane is unfavorable (for instance the runway beam and the column). The number of the alternative crane

Table 6.4. Groups of loads considered as one characteristic crane action and corresponding dynamic factors (EN 1991-3)

	Symbol[1)]	*Section*[1)]	*Groups of loads*									
			Ultimate Limit State							Test load	Accidental	
			1	2	3	4	5	6	7	8	9	10
1 Self-weight of crane	Q_c	2.6	ϕ_1	ϕ_1	1	ϕ_4	ϕ_4	ϕ_4	1	ϕ_1	1	1
2 Hoist load	Q_h	2.6	ϕ_2	ϕ_3	-	ϕ_4	ϕ_4	ϕ_4	$\eta^{2)}$	–	1	1
3 Acceleration or crane bridge	H_L, H_T	2.7	ϕ_5	ϕ_5	ϕ_5	ϕ_5	–	–	–	ϕ_5	–	–
4 Skewing of crane bridge	H_S	2.7	–	–	–	–	1	–	–	–	–	–
5 Acceleration of braking of crab or hoist block	H_{T3}	2.7	–	–	–	–	–	1	–	–	–	–
6 In-service wind	F_W	Annex A	1	1	1	1	1	–	–	1	–	–
7 Test load	Q_T	2.10	–	–	–	–	–	–	–	ϕ_6	–	–
8 Buffer load	H_B	2.11	–	–	–	–	–	–	–	–	ϕ_7	–
9 Tilting force	H_{TA}	2.11	–	–	–	–	–	–	–	–	–	1

[1] Symbols and sections indication as in EN 1991-3, 2006.
[2] η is the proportion of the hoist load that remains when the payload is removed, but is not included in the self-weight of the crane.

Table 6.5. Recommended maximum number of cranes to be considered in the most unfavourable position (EN 1991-3)

	Cranes to each runway	*Cranes in each* shop bay	*Cranes in multi-bay* buildings	
				
Vertical crane action	3	4	4	2
Horizontal crane action	2	2	2	2

actions increase further when many cranes operate in the same building. Designers should select the necessary number of crane actions, according to their engineering judgment, in order to obtain critical loading for all different structural members.

6.6.3 Ultimate limit states

6.6.3.1 General

Crane supporting beams are subjected to:

a) Bending moments around the strong axis of the cross-section due to vertical loads
b) bending moments around the weak axis due to horizontal lateral loads

c) shear forces related to the aforementioned bending
d) axial forces due to the longitudinal components of the horizontal forces
e) torsion moments due to the eccentricity of both vertical (Fig. 6.71a) and horizontal forces, which apply to the top of the rails, in respect to the shear center of the beams cross-section

In addition local stresses develop under the concentrated wheel loads.

As the above forces and moments constitute a relatively complex loading system, EN 1993-6, in the sake of a simplification, allows following assumptions to be made, except when box sections are used as runway beams:

a) vertical loads are resisted by the overall beam cross-section,
b) horizontal loads, applied to the level of the upper flange, are resisted by the cross -section of this flange only,
c) torsion moments can be decomposed to couples of forces acting to the beams at the levels of the upper and lower flange centers.

6.6.3.2 Local stresses in the web of the runway beams

In the usual case where rails are fixed on the top flange of the runway beam, local stresses develop in its web due to the concentrated wheel loads. It is to be distinguished between: (a) local vertical stresses (b) local shear stresses and (c) local bending stresses due to the eccentricity of the wheel vertical loads (see 6.6.2).

The distribution of vertical local compressive stresses under the wheel load, is shown in the figure of Table 6.6. The distribution width depends on the rail's shape, dimensions and rigidity, the thickness and rigidity of the crane beam upper flange, the way of fixing the rails to the top flange and the thickness of the web. To simplify calculations, EN 1993-6 allows to assume a uniform stress distribution on an effective length measured on the lower surface of the top flange. To determine this effective length the previously mentioned factors are taken into account as shown in Table 6.6 where t_w is the web thickness, I_r the second moment of area of the rail about its own centroidal axis and $I_{f,eff}$ the second moment of area of an effective width of the top flange. When the rail is rigidly fixed to the top flange, I_{rf} is the second moment of area of the combined cross-section consisting of the rail and the effective width of the top flange around the centroidal axis of this complex cross-section. The effective width of the top flange is calculated according to:

$$b_{eff} = b_{fr} + h_r + t_f \leq b \tag{6.11}$$

where b_{fr} is the width of the contact surface between the rail and the top flange, h_r is the height of the rail, t_f is the top flange thickness and b is the overall width of the flange.

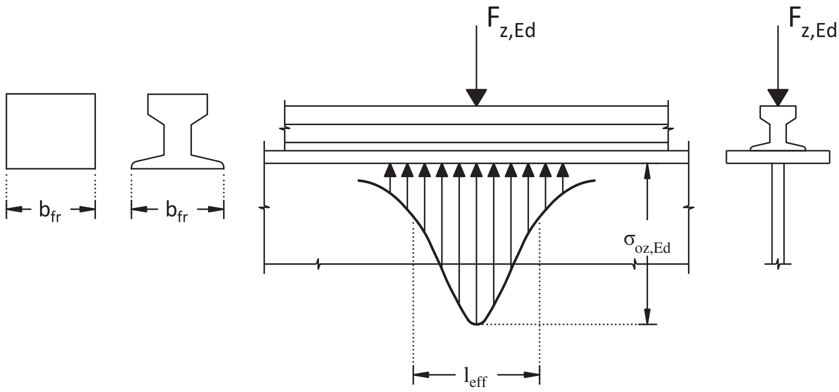
When introducing h_r, I_r, I_{rf}, wear of the rail equal to the 25% of the minimum nominal thickness t_r below the wearing surface should be considered (see Fig. 6.69). The design value of the local vertical compressive stress at the lower surface of the top flange could then be calculated as:

$$\sigma_{z,Ed} = \frac{F_{z,Ed}}{l_{eff} \cdot t_w} \tag{6.12}$$

where $F_{z,Ed}$ is the design value of the wheel vertical load.

Table 6.6. Effective loaded length l_{eff} (EN 1993-6)

Case Description		Effective loaded length, l_{eff}
(a)	Crane rail rigidly fixed to the flange	$l_{eff} = 3.25[I_{rf}/t_w]^{1/3}$
(b)	Crane rail not rigidly fixed to flange	$l_{eff} = 3.25[(I_{rf} + I_{f,eff})/t_w]^{1/3}$
(c)	Crane rail mounted on a suitable resilient elastomeric bearing pad at least 6mm thick	$l_{eff} = 4.25[(I_{rf} + I_{f,eff})/t_w]^{1/3}$



Under the above level, a 45° stress distribution is allowed to be considered, therefore for a hot rolled beam the local stress on the top of the web, of thickness t_w, can be calculated with an effective length of $l_{eff} + 2r$, where r is the radius between web and flange. If the distance between the centers of two adjacent crane wheels is less than l_{eff}, which is not the usual case, the local stresses from the two wheels should be superimposed. At the supports the web of the runway beams is usually reinforced by transverse stiffeners.

Due to the non-uniform distribution of the vertical compression stresses, as above, shear stresses develop in the web, in addition to those resulting from the applied shear forces. Following the EN 1993-6 rules, the additional shear stresses, acting on both sides of the wheel load position, may be assumed as equal to 20% of the maximum value of the local vertical compressive stress. The additional shear stresses develop only near the point of application of the concentrated force, at the top flange, and may be neglected at a distance greater than $0.20h_w$, where h_w is the overall depth of the web.

Local bending stresses also develop in the web of runway beams due to the eccentricity e_y (Fig. 6.71) of the vertical wheel load $F_{z.Ed}$ and the corresponding torsional moment $T_{Ed} = F_{z.Ed}e_y$. Following EN 1993-6, in the case of a beam with transverse stiffeners, the maximum value of this stress may be determined using the theoretical equation from elasticity:

$$\sigma_{T,Ed} = \frac{6 \cdot T_{Ed}}{a \cdot t_w^2} \cdot n \cdot \tanh(n) \tag{6.13}$$

with

$$n = \left[\frac{0.75 \cdot a \cdot t_w^3}{I_t} \cdot \frac{\sinh^2(\pi \cdot h_w/a)}{\sinh(2 \cdot \pi \cdot h_w/a) - \dfrac{2 \cdot \pi \cdot h_w}{a}} \right] \tag{6.14}$$

where: a is the spacing of the transverse web stiffeners. Usually stiffeners are placed only at the supports. h_w is the clear depth of the web, between the flanges, I_t is the torsional constant of the flange (including the rail if it is rigidly fixed on the beam).

It is also recommended to maintain, in any case, $e_y \geqslant 0.5t_w$(t_w the web thickness).

6.6.3.3 Cross-section verification

The cross-section verification can be performed using appropriate interaction formulae, based on the design strength of the partial resistances, or, more practically, in terms of stresses, by calculating in the critical points of the most loaded cross-sections the von Mises equivalent stress. This stress must be checked to be less than the yield stress, see equation (3.49). Direct and shear stresses due to bending moments around the strong and the weak axis of the cross-section, as well as torsional moments arising from the eccentricity of both vertical and coexistent horizontal forces, together with local stresses as previously presented, have to be considered in the calculation of the von Mises stress.

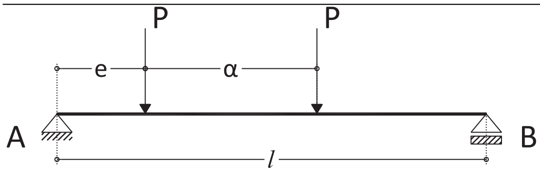
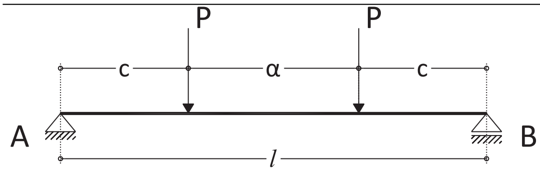
Runways are composed of successive simply supported beams between adjacent columns. The maximum bending moment in such a beam with span l, subjected to a couple of moving loads P, at a spacing a, is presented in Table 6.7. When the spacing a is larger than 0.586l, the maximum moment develops when one load is in the middle of the span. The beam is also to be checked against shear forces. For this verification the most critical position for the loads P is when the first acts over the support of the beam. Shear buckling of the web is usually not a critical mode of failure, as the web is relatively thick to resist local vertical stresses. In any case plate buckling should be checked in accordance with EN 1993-1-5 [6.16].

According to EN 1993-6, the rail could be considered as part of the overall cross-section if it is rigidly fixed on the runway beam. In this case a rail wear of 25%, as indicated previously, should also been considered.

6.6.3.4 Member verification

Runway beams, as members subjected mainly to bending about the major axis, should be checked against lateral-torsional buckling. Interaction or other formulae in the regulations, established to verify structural members under bending against lateral-torsional buckling (EN 1993-1-1 and chapter 4), are not valid in case of coexistence of torsional moments, and cannot, therefore, be applied for crane supporting beams. As this verification could be critical for the beam, it is usual to arrange, at the level of its top flange, a horizontal structural element (for instance a truss) offering a lateral support and restraining horizontal deformations of the compression flange (see also 6.6.7). This restrain element is usually called a surge girder, as it transfers, due to its rigidity, crane surge (see 6.6.2) to the supports.

Table 6.7. Maximum bending moment and deflection of a simply supported beam subjected to a pair of equal loads

	$\max M = \frac{P}{8l}(2l - a)^2$ for $e = \frac{2l - a}{4}$ and $\alpha < 0.586 \cdot l$
	$\max f = \frac{Pc}{24El}(3l^2 - 4c^2)$ for $\alpha < 0.65 \cdot l$

For laterally unprotected beams EN 1993-6 includes, in Annex, an interaction formula in which both bending and torsional moments are introduced. In addition it provides an alternative simplified design procedure according to which an equivalent flexural buckling verification, in respect to the vertical, weak, axis of the cross-section, is performed in an ideal member, of the same length, subjected to an axial compression force. This member has a T-cross-section consisting of the upper flange of the beam plus 1/5 of the web. The axial force is calculated on the basis of the maximum bending moment divided by the distance between centers of the two flanges and it is considered to act together with the coexistent horizontal forces applied at the upper flange of the beam.

6.6.4 Serviceability limit states

To ensure a smooth and free movement of the crane bridge and limit wearing and the possibility of a derailment, the deformability of both, crane supporting beam and building, should be within appropriate limits. To this end it should be limited:

a) the vertical deformation of the runway beam to avoid excessive slope of the rail, as well as excessive beam vibrations during hoisting of the load or crane operation
b) the differential vertical deformation of the pair of runway beams to avoid excessive slope of the crane bridge
c) the horizontal deformation of the beams to reduce skewing consequences
d) the lateral displacement of a column at the crane support level, to avoid excessive frame vibrations

e) the differential lateral horizontal displacement of adjacent columns, at the level of the crane support, to avoid misalignment of the rails which increases skewing forces and induces a possible distortion of the crane bridge
f) the lateral deformations which can change the spacing between the two supporting beams in order to avoid damages and wear to the wheels flanges or rail fixings or a derailment of the crane.

As application rules the limits recommended in EN 1993-6 are indicated in Table 6.8 for the runway beams and in Table 6.9 for the building. The last limitation of 10 mm in this Table is a demanding requirement, governing, frequently, the dimensions of the frames. In such cases the note at the end of the Table has to be considered. The above limits, together with the serviceability combinations under which they apply, should be agreed, for each specific project, between the crane supplier and the client. National Annexes may also specify limit values different from the above. EN 1993-6 recommends the limits included in the Tables to be checked for the characteristic combination of actions. It is also recommended to consider vertical deflections without any dynamic amplification factor. In any case stresses and displacements at the serviceability limit states should be calculated using a linear elastic analysis.

For a simply supported beam with span l and rigidity EI, subjected to a pair of equal vertical loads P at a spacing a, the maximum deflection and the corresponding position of loads, are indicated in Table 6.7. When the spacing a is greater than 0.65l the maximum deflection is obtained when the first load applies at the middle of the span.

Table 6.8. Limit deformation values for the runway beams (EN 1993-6)

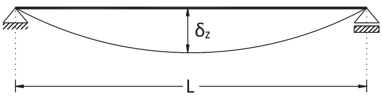
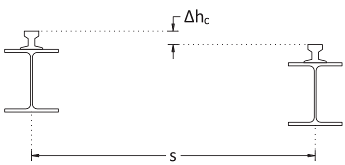
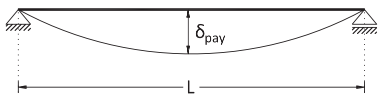
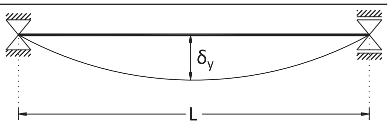
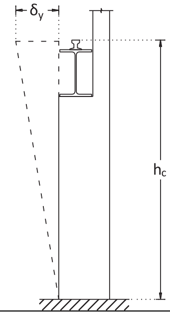
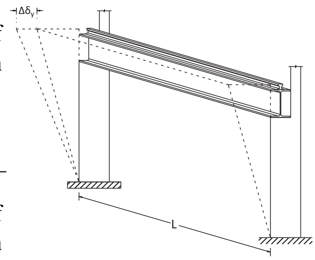
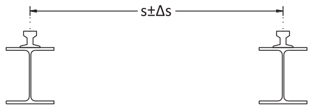
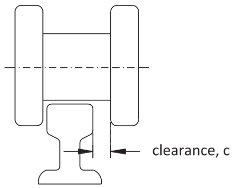
Description of deflection (deformation or displacement)	Diagram
a) Vertical deformation δ_z of a runway beam: $\delta_z \leq L/600$ and $\delta_z \leqslant 25$ mm. The vertical deformation δ_z should be taken as the total deformation due to vertical loads, less the possible pre-camber, as for $\delta_{\max}$ in Figure 1.1 of EN 1990.	
b) Difference Δh_c between the vertical deformations of two beams forming a crane runway: $\Delta h_c \leqslant s/600$	
c) Vertical deformation δ_{pay} of a runway beam for a monorail hoist block, relative to its supports, due to the payload only: $\delta_{pay} \leqslant L/500$	
d) Horizontal deformation δ_y of a runway beam, measured at the level of the top of the crane rail: $\delta_y \leqslant L/600$	

Table 6.9. Limit displacements values for the buildings (EN 1993-6)

Description of deflection (deformation or displacement)	Diagram
a) Horizontal displacement δ_y of a frame (or of a column) at crane support level, due to crane loads: $\delta_y \leqslant h_c/400$ where: h_c is the height to the level at which the crane is supported (on a rail or on a flange)	
b) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent frames (or columns) supporting the beams of an indoor crane runway: $\Delta\delta_y \leqslant L/600$	
c) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent columns (or frames) supporting the beams of an outdoor crane runway: - due to the combination of lateral crane forces and the in-service wind load: $\Delta\delta_y \leqslant L/600$ - due to the out-of-service wind load $\Delta\delta_y \leqslant L/400$	
d) Change of spacing Δs between the centres of crane rails, including the effects of thermal changes: $\Delta s \leqslant 10$ mm [see Note]	
NOTE: Horizontal deflections and deviations of crane runways are considered together in crane design. Acceptable deflections and tolerances depend on the details and clearances in the guidance means. Provided that the clearance c between the crane wheel flanges and the crane rail (or between the alternative guidance means and the crane beam) is also sufficient to accommodate the necessary tolerances, larger deflection limits can be specified for each project if agreed with the crane supplier and the client.	

6.6.5 Fatigue

In general, for the structural elements in steel buildings, fatigue is not considered, as there is very limited possibility, during the life time of the building, to reach a very important number of loading cycles, at a high level of loading, due to the usual variable actions (live load, wind, snow). An exception from this rule are crane supporting beams, which are destined, due to their operational role, to support at a high load level many cycles of loading. It is evident that the sensitivity of a runway beam against fatigue depends on the total number of loading cycles, during its life time, the frequency of hoisting loads with a value close to the crane capacity as well as on the type of the crane (operational conditions, hoisting system).

The most sensitive area against fatigue for H-beams is the region where flange and web come together. Hot-rolled cross-sections are less sensitive compared to built-up sections in which the web is welded to the flanges. For built-up sections there is also an important difference on the fatigue resistance, between the full penetration butt welds and fillet welds. In fillet welds there is not, before welding, a continuous contact between the flange and the web. The existing small void, in many places along the beam, provides space for deformations each time the wheel passes over the void. Following this effect, the welds are subjected, besides the main stresses, to cycles of additional deformation and stresses and therefore to fatigue conditions. The deformations caused by the voids, cannot be develop in full penetration butt welds, as the contact between flanges and web is continuous.

Fatigue loading should be determined on the basis of the crane operational conditions, mainly the distribution of loading during the lifetime of the crane (number of hoists at each level of loading). In this case a verification following the Miner-Palmgren rule could be performed. As, during the design stage, sufficient information is not frequently available, EN 1993-6 proposes a simplified procedure to check crane supporting beams against fatigue, based on the provisions of EN 13001 [6.14, 6.15] and EN 1993-1-9: "Design of steel structures. Fatigue" [6.17]. According to this procedure:

a) cranes are classified, following their sensitivity to a possible fatigue damage, in ten categories (S0, S1, ... , S9) depending on the total number of loading cycles, during a total service life of 25 years and the relation of these cycles to the nominal hoisting capacity of the crane, as shown in Table 6.10. More information about this classification is included in EN 13001-1 [6.14]. The crane class could be given by the crane supplier. In the absence of any information EN 1991-3 includes in an Annex a guidance Table (see Table 6.11) for an indicative crane classification for fatigue, depending on the type and the location where the crane works. For instance, power house cranes or erection cranes with hook operation are classified in this Table in the S1 or S2 classes, workshop cranes or shipboard cargo cranes with hook operation in classes S3 or S4, storage cranes with continuous operation or overhead travelling cranes with grab or magnets in classes S6 or S7, stripper or charging cranes in classes S8 or S9. In the same Table the classification of the cranes on hoisting classes (see 6.6.2) is also included.
b) An ideal fatigue damage equivalent load Q_c is determined which, applied with its constant value for $2 \cdot 10^6$ cycles of loading is considered to be equivalent with the real fatigue loading history of the crane. This fatigue load is determined from:

$$Q_c = \phi_{\text{tot}} \cdot \lambda_i \cdot Q_{\text{max}} \tag{6.15}$$

Table 6.10. Classification of cranes in fatigue classes – Values of coefficient λ and the partial safety factor for the design value of the fatigue strength (EN 1991-3 and EN 1993-6)

Class of load spectrum		Q_0	Q_1	Q_2	Q_3	Q_4	Q_5
		$kQ \leqslant 0.0313$	$0.0313 < kQ \leqslant 0.0625$	$0.0625 < kQ \leqslant 0.125$	$0.125 < kQ \leqslant 0.25$	$0.25 < kQ \leqslant 0.5$	$0.5 < kQ \leqslant 1.0$
class of total number of cycles							
U_0	$C \leqslant 1.6 \cdot 10^4$	S_0	S_0	S_0	S_0	S_0	S_0
U_1	$1.6 \cdot 10^4 < C \leqslant 3.15 \cdot 10^4$	S_0	S_0	S_0	S_0	S_0	S_1
U_2	$3.15 \cdot 10^4 < C \leqslant 6.30 \cdot 10^4$	S_0	S_0	S_0	S_0	S_1	S_2
U_3	$6.30 \cdot 10^4 < C \leqslant 1.25 \cdot 10^5$	S_0	S_0	S_0	S_1	S_2	S_3
U_4	$1.25 \cdot 10^5 < C \leqslant 2.50 \cdot 10^5$	S_0	S_0	S_1	S_2	S_3	S_4
U_5	$2.50 \cdot 10^5 < C \leqslant 5.00 \cdot 10^5$	S_0	S_1	S_2	S_3	S_4	S_5
U_6	$5.00 \cdot 10^5 < C \leqslant 1.00 \cdot 10^6$	S_1	S_2	S_3	S_4	S_5	S_6
U_7	$1.00 \cdot 10^6 < C \leqslant 2.00 \cdot 10^6$	S_2	S_3	S_4	S_5	S_6	S_7
U_8	$2.00 \cdot 10^6 < C \leqslant 4.00 \cdot 10^6$	S_3	S_4	S_5	S_6	S_7	S_8
U_9	$4.00 \cdot 10^6 < C \leqslant 8.00 \cdot 10^6$	S_4	S_5	S_6	S_7	S_8	S_9

Values of λ										
categories	S_0	S_1	S_2	S_3	S_4	S_5	S_6	S_7	S_8	S_9
normal stresses	0.198	0.250	0.315	0.397	0.500	0.630	0.794	1.000	1.260	1.587
shear stresses	0.379	0.436	0.500	0.575	0.660	0.758	0.871	1.000	1.149	1.320

Values of the partial safety factor for the fatigue strength		
Assessment method	Consequences of failure	
	Low consequence	High consequence
Damage tolerant	1.00	1.15
Safe life	1.15	1.35

where: $Q_{\max}$ is the maximum value of the characteristic vertical wheel load, λ_i is a damage equivalent factor through which the above classification into fatigue classes is taken into account, and ϕ_{tot} is the damage equivalent dynamic impact factor taken equal to $(1+\phi_1)/2$ and $(1+\phi_2)/2$ for the two parts of Q_{max} corresponding to the self-weight of the crane and the hoist load respectively.

The values of factor λ are included in Table 6.11. Two different values for λ_i are provided to calculate the equivalent fatigue load corresponding to direct and shear stresses. On the basis of the above equivalent fatigue load the stress variation due to one cycle of loading is calculated.

c) The fatigue resistance of each structural element or each connection is provided in detailed Tables in the related to the fatigue EN 1993-1-9. The different details are classified in ten categories the names of which correspond to the fatigue strength. In this way a detail classified in category 71 can nominally resist a stress variation of 71 MPa for $2 \cdot 10^6$ cycles. The Table corresponding to the web

Table 6.11. Guidance for crane classification for fatigue (EN 1991-3)

Item	*Type of crane*	*Hoisting class*	*S-classes*
1	Hand-operated cranes	HC1	S0, S1
2	Assembly cranes	HC1, HC2	S0, S1
3	Powerhouse cranes	HC1	S1, S2
4	Storage cranes – with intermittent operation	HC2	S4
5	Storage cranes, spreader bar cranes, scrap yard cranes – with continuous operation	HC3, HC4	S6, S7
6	Workshop cranes	HC2, HC3	S3, S4
7	Overhead travelling cranes, ram cranes – with grab or magnet operation	HC3, HC4	S6, S7
8	Casting cranes	HC2, HC3	S6, S7
9	Soaking pit cranes	HC3, HC4	S7, S8
10	Stripper cranes, charging cranes	HC4	S8, S9
11	Forging cranes	HC4	S6, S7
12	Transporter bridges, semi-portal cranes, portal cranes with trolley or slewing crane – with hook operation	HC2	S4, S5
13	Transporter bridges, semi-portal cranes, portal cranes with trolley or slewing crane – with grab or magnet operation	HC3, HC4	S6, S7
14	Travelling belt bridge with fixed or sliding belt(s)	HC1	S3, S4
15	Dockyard cranes, slipway cranes, fitting-out cranes – with hook operation	HC2	S3, S4
16	Wharf cranes, slewing, floating cranes, level luffing slewing – with hook operation	HC2	S4, S5
17	Wharf cranes, slewing, floating cranes, level luffing slewing – with grab or magnet operation	HC3, HC4	S6, S7
18	Heavy duty floating cranes, gantry cranes	HC1	S1, S2
19	Shipboard cargo cranes – with hook operation	HC2	S3, S4
20	Shipboard cargo cranes – with grab or magnet operation	HC3, HC4	S4, S5
21	Tower slewing cranes for the construction industry	HC1	S2, S3
22	Erection cranes, derrick cranes – with hook operation	HC1, HC2	S1, S2
23	Rail mounted slewing cranes – with hook operation	HC2	S3, S4
24	Rail mounted slewing cranes – with grab or magnet operation	HC3, HC4	S4, S5
25	Railway cranes authorized on trains	HC2	S4
26	Truck cranes, mobile cranes – with hook operation	HC2	S3, S4
27	Truck cranes, mobile cranes – with grab or magnet operation	HC3, HC4	S4, S5
28	Heavy duty truck cranes, heavy duty mobile cranes	HC1	S1, S2

to flange joints in crane supporting beams is reproduced in Table 6.12. It is to be noted that the fatigue resistance to vertical compressive stresses in the web is 160 MPa for hot-rolled beams, 71 MPa for built-up beams with full penetration butt welds and 36 MPa for built-up beams with fillet welds. It is clear that the latter type of welding in fatigue sensitive runway beams is to be avoided.

d) The verification against fatigue is performed using the following formulae corresponding to vertical compressive stresses, shear stresses and the interaction between them.

$$\frac{\gamma_{Ff} \cdot \Delta\sigma_{E,2}}{\Delta\sigma_c/\gamma_{Mf}} \leq 1.0 \quad (6.16)$$

$$\frac{\gamma_{Ff} \cdot \Delta\tau_{E,2}}{\Delta\tau_c/\gamma_{Mf}} \leq 1.0 \quad (6.17)$$

$$\left(\frac{\gamma_{Ff} \cdot \Delta\sigma_{E,2}}{\Delta\sigma_c/\gamma_{Mf}}\right)^3 + \left(\frac{\gamma_{Ff} \cdot \Delta\tau_{E,2}}{\Delta\tau_c/\gamma_{Mf}}\right)^5 \leq 1.0 \quad (6.18)$$

The numerators are the fatigue design stress ranges calculated on the basis of the ideal equivalent fatigue loads. The partial safety factor γ_{Ff} is recommended to be taken equal to 1.0. National application documents could adopt different values. The denominators correspond to the related fatigue resistances obtained from the Tables of EN 1993-1-9 (as for example Table 6.12). The partial safety factor γ_{Mf}, which adapts the nominal fatigue strength to a design value, is recommended to have the values included in Table 6.10 depending on the assessment of a damage tolerant or a safe life structure and on the level of the consequences of a failure.

In addition to the above in EN 1993-6 it is noted that:

a) A fatigue verification is required for the components of the supporting structures that are subjected to stress variations from vertical loads. Stress variations due to horizontal loads are not, in the usual cases, significant.
b) Local stresses should be considered in the fatigue assessment.
c) In the web the compressive local stresses under the wheel loads, the related shear stresses and the bending stresses due to the eccentricity of the wheel loads should be introduced in the verification.
d) The above local bending stresses could be neglected for cranes classified in S0 to S3 classes.
e) If the total number of loading cycles exceeding 50% of the nominal capacity of the crane is less than $C=10^4$, the fatigue assessment could be omitted. National Annexes could adopt a different value for C.
f) When the rail is rigidly connected to the beam and is considered in the calculations, as part of the runway beam cross-section, half of the rail wear mentioned in 6.6.3 should be taken into account in the fatigue verifications.

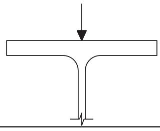
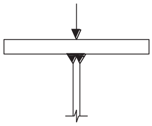
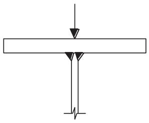
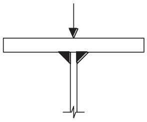
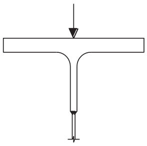
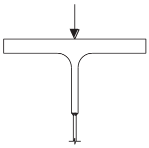
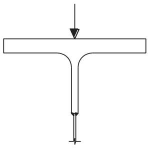
Finally in EN 1993-6 information related to crane supporting beams loaded by two or more cranes (multiple crane action) is given.

6.6.6 Specific verifications

In the design of crane supporting beams some additional verifications should be considered concerning: (a) reversibility of stresses (b) breathing of the web and (c) vibration of the lower flange.

As runway beams should fulfill serviceability and fatigue limitations it is advisable to avoid even limited plastic deformations. To this end it is recommended that

Table 6.12. Top flange to web junction of runway beams (EN 1993-1-9)

Detail category	Constructional detail	Description	Requirements
160		1) Rolled I- or H-sections	1) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
71		2) Full penetration tee-butt weld	2) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
36*		3) Partial penetration tee-butt welds, or effective full penetration tee-butt weld conforming with EN 1993-1-8	3) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
36*		4) Fillet welds	4) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
71		5) T-section flange with full penetration tee-butt weld	5) Vertical compressive stress range $\Delta\sigma_{vert.}$ in web due to wheel loads
36*		6) T-section flange with partial penetration tee-butt weld, or effective full penetration tee-butt weld conforming with EN 1993-1-8	6) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads
36*		7) T-section flange with fillet welds	7) Stress range $\Delta\sigma_{vert.}$ in weld throat due to vertical compression from wheel loads

stresses developed during the service loading remain in the elastic range, in other terms that stresses under a serviceability load combinations are reversible.

To this intention EN 1993-6 requires, as an application rule, that the von Mises equivalent stress, due to the characteristic serviceability combination, is smaller, at any point of the beam, than the yield stress, equation (3.49). It is evident that when the cross-section verification is performed in terms of stresses (see 6.6.3) the reversibility required is ensured. To the same objective National Annexes could adopt a different limit stress, than first yielding. The reversibility should also be ensured under the test loading conditions. In case of a beam with a wide upper flange allowance should be considered for the effects of shear lag. Local stresses are to be also introduced in the von Mises combined stress, whereas only bending stresses in the web due to vertical loading eccentricity could be neglected.

When web panels in the runway beams are relatively slender, breathing of this web could be take place during the in service loading of the beam. This type of instantaneous elastic buckling of the web has to be avoided as it results fatigue phenomena mainly in the web to flange welding [6.18].

EN 1993-6 offers an interaction direct-shear stresses formula, based on the linear elastic buckling coefficients given in EN1993-1-5 [6.16] and EN 1993-2 [6.19], the verification of which ensures the limitation of web buckling. The formula corresponds to panels assumed to have hinge edges. Stresses introduced in this formula correspond to the frequent load combination of EN 1990, meaning that this verification belongs to the serviceability requirements. As a simplified rule, according to EN 1993-6, excessive web buckling is avoided in panels without longitudinal stiffeners, when the ratio b/t_w in the web is less than 120, where b is the smaller dimension of the web panel and t_w the web thickness. Breathing concerns more the design of the main beams in bridges and, in the usual cases, it is not a critical limitation for the runway beams in buildings.

Finally the possibility of lateral vibrations of the runway beam bottom flange during crane operation should be avoided. This vibration could be probably appear in beams of mono-symmetric cross-section with a wide top flange (which gives more space for the rails fixing and improves the strength against lateral-torsional buckling) and a narrow lower flange. To avoid this vibration EN 1993-6 recommends, as a simple application rule, to limit the ratio L/i_z in a value less than 250, where L is the length of the lower flange, between lateral restraints, and i_z the radius of gyration of this flange about the vertical axis z.

6.6.7 Conceptual design. Constructional details

The connection of a runway beam to the typical frame should be realized in such a way to ensure (a) that the cross-section at the support cannot twist in order to fulfill the assumptions for the verification against lateral-torsional buckling and (b) that the beam is free to develop vertical and horizontal deformations as well as the corresponding end rotations. Indicative details are presented in Fig. 6.73. Runway beams are usually bolted to the supporting elements through their lower flange. To avoid the development of a couple of forces during the beam deformation and a not anticipated tension in the bolts, additional plates, as in Fig. 6.74, are recommended to be arranged, especially for beams supporting cranes with higher hoisting capacities.

For welded runway beams intermittent fillet welds should not be used for the web to flange welding to ensure, in all sections, a continuous transmission of forces from the wheels to the web. Intermittent fillet welds are also not recommended for the crane supporting structural elements as they are sensitive to a fatigue failure and may lead to the creation of rust pockets. For high fatigue crane classes (S7 to S9) it is recommended that web stiffeners at the support positions should not be welded to the top flanges of the runway beams.

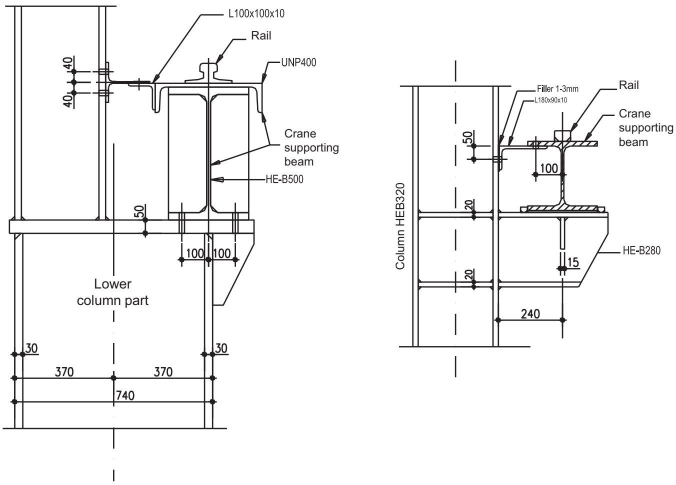


Fig. 6.73. Supporting positions of crane supporting beams. Surge connections

For the selection of the type of rail one should consider the rail material, the wheel diameter and wheel material, the wheel load and the crane utilization. For the selected cross-section of the rail, the contact pressure between rail and wheel should be limited to reduce friction and excessive wear of both the rail and wheel. A design method is given in EN 13001-3-3 [6.20]. The rails could be continuous over the joints of the runway beams or discontinuous with expansion joints. Rail joints, when arranged, should be designed to minimize impact. They should be oblique to the rail direction at a distance from the runway beams ends. According to Eurocodes, purpose made crane rails should be made of special rail steels with a specified minimum tensile strength between 500 and 1200 MPa.

The fixings of the rails on the runway beams could be classified as rigid or independent. Connections through welding or through preloaded or fit bolts passing through the flange of the rail are classified as rigid. The classification of the connections as rigid requires that they are able to resist against longitudinal forces between the rail and the beam, due to bending, in addition to the lateral forces applied by the wheels. They should also be checked against fatigue. Independent fixings are mainly clamps, placed at both sides of the rail, with a suitable spacing to resist the above lateral forces. A rail with independent fixings could have a suitable resilient elastomeric bedding material (between rail and beam) to reduce dynamic effects and improve stresses distribution under the wheel.

In EN 1090-2 [6.3] limits for some constructional imperfections, related to the crane supporting beams and the rails are included. The above limits, named functional tolerances, should be respected to meet functions other than resistances and stability as well as strength and rigidity of the elements. Reference to them is included in 8.1 and 8.7.

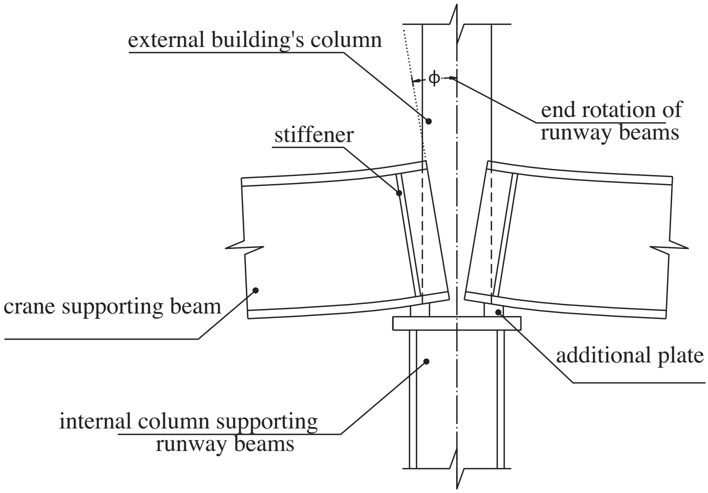


Fig. 6.74. End rotation of runway beams

6.6.8 Underslung cranes

In underslung cranes the crane wheels move along the lower flange of the runway beams (on both sides of the web). This arrangement could be selected, for instance, in the case of a building with a high clear height and a crane with low hoisting capacity. An example of such a building is shown in Fig. 6.75 where the underslung crane moves on the lower flange of four runway beams. In such cases the runway beams should resist, besides the general stresses, additional local bending stresses on their lower flanges in the vicinity of the wheel load. In the lower flange of the runway beam are also supported the hoists of monorails.

EN 1993-6 gives a detailed method to calculate the aforementioned local stresses for both cases of beams with tapered or of a constant thickness flange, as well as for an intermediate place of the beam or on its ends, where local stresses take higher values. The values of the local stresses (Fig. 6.76) are given at points 0 (web to flange transition), 1 (under the wheel load) and 2 (outside edge of the flange). To avoid the selection of a beam with a thicker lower flange, due to the high values of local stresses at the ends of the beam, it is recommended, as an alternative, to weld on this end part additional reinforcing plates (see Fig. 6.76). In addition, a detailed procedure is also proposed to determine the strength $F_{f.Rd}$ of the bottom flange in relation with the

Fig. 6.75. Underslung crane on four runway beams in an airplanes hangar

normal stresses due to overall bending. This strength should be compared with the wheel load $F_{z.Ed}$ (Fig. 6.76).

For underslung cranes the horizontal forces at the wheel contact surface should be taken, according to EN 1991-3, equal to 10% of the maximum vertical wheel load, where in this value it is considered that the dynamic effect is included. In fixed runway beams for monorail underslung trolleys, in the absence of a more accurate value, the longitudinal horizontal force could be taken equal to 5% of the maximum vertical wheel load without an additional dynamic factor. The same value could also apply for horizontal loads in swinging suspended monorail beams. Finally, to verify the reversibility of stresses both overall and local stresses should be considered.

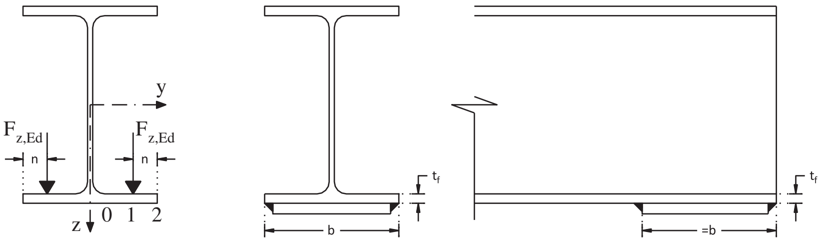


Fig. 6.76. Underslung cranes. Local stresses on the bottom flange. Reinforcement of the end part of the runway beam

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7
Multi storey buildings

Abstract. The chapter presents the structural elements of steel multi storey buildings, such as columns, main and secondary beams or concrete slabs. For these elements various cross-section types, both pure steel or composite steel-reinforced concrete, are shown and alternative options commented, presenting the advantages and characteristics of each one. Information is included in respect to the serviceability requirements, the behavior under fire conditions and the methods of construction.

It follows the presentation of the various structural systems that ensure the overall lateral stability of the buildings, such as moment resisting frames (MRF), concentric (CBF) or eccentric (EBF) bracing systems, shear walls of different types or combination of the above, in association with the diaphragm action of the floor slabs and the rigidity of the connections and joints (simple, rigid or semi-rigid joints). This enables the designer to obtain sufficient information to select, for each specific case of a building, the appropriate structural configuration for the overall structure, and for its parts, such as the form and cross-section type of the individual structural elements and the connections and joints.

The chapter concludes with the presentation of the provisions of EN 1998 (Eurocode 8) for buildings constructed in seismic regions. These rules are specific for each type of system ensuring the overall lateral stability of the structure (MRF, CBF, EBF etc.) and are presented in specific sections. The seismic rules are related to the required stiffness and strength, the hierarchy of yielding, known as capacity design, as well as the damage limitation for the non-structural elements of the building in cases of frequent earthquakes, weaker than the design ones. The above seismic rules are to be considered in combination with the corresponding ones, presented in relevant section of chapter 6.

7.1 Introduction

The use of structural steel started during the 19^{th} century for the construction of bridges, roofs, as coverings of areas, and simple single storey buildings. However, from the beginning of the 20^{th} century, steel started to be used as skeleton of higher multi storey buildings in the USA. The very rapid industrial development, especially in the mid-western States, lead to increased needs for offices, commercial shops or warehouses. These increased constructional needs, in combination with the higher values of the land, especially in cities like Chicago, required extension of buildings

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in the elevation and shortening of the construction time, therefore the quest for new constructional methods and materials. Steel presented many structural advantages, as strength, flexibility and very short erection times, and therefore became the appropriate solution compared with the traditional constructional methods. Under these circumstances a large number of steel buildings were constructed, mainly in Chicago and New York, using steel frames as skeleton, concrete slabs to receive vertical loads and brick walls for their lateral stability.

Similar situations are in our times presented in North-East Asia and especially in China. The development of big urban centers in Shanghai, Hong-Kong or Guangzhou (Canton) is associated with the increasing needs for multi storey buildings, the majority of which are built with steel as the main structural material. However, the most extensive use of steel buildings is in Japan, due to the excessive seismicity of the area, where more than half of the multi storey buildings are built using steel.

Steel buildings nowadays are noticeable for their large variety of shapes, the large spans, the natural lighting and the overall impression they give, as modern and aesthetic constructions. They apply mainly for office buildings, banks, hotels, commercial centers, garages. Applications of steel multi storey buildings in Europe are less compared to USA and Japan. Extensive use of steel buildings is noticed in London and the Scandinavian countries. Steel medium rise buildings offer important advantages, compared to concrete buildings, in many fields such as high degree of industrial prefabrication, with the related positive consequences on the quality assurance, shortening of the construction time, lower mass of the structure and the related advantages for foundations, very satisfactory seismic behavior due to the ductility of steel and the reduced masses of the structure, smaller cross-sections for the structural members (beams, columns), facility in structural strengthening, modifications and additions, lower sensitivity to environmental conditions, possibility to disconnect members and dismantling the structure. On the other hand steel buildings are more sensitive against corrosion of their external members as well as against fire conditions, more sensitive to vibrations and more difficult to collaborate with brittle materials.

In Figure 7.1 indicative examples of multi storey steel buildings are shown, constructed in Athens for the International Television Center and the Main Press Center, operated during the Athens Olympic Games, 2004.

7.2 Main structural elements of a steel multi storey building

7.2.1 General

The main elements that compose the structural system of a multi storey steel building are columns, main beams, secondary beams, slabs and vertical bracing systems, if any. At the start of the design the columns' grid is fixed. The main beams join, at the levels of the successive floors, the column heads, usually in all axes, in both main directions of the building. In this way a 3D frame is formed. In each horizontal panel of the grid secondary beams are provided, oriented to one direction. Consequently among the main beams, which determine the panel, those supporting the secondary

(a)
(b)

Fig. 7.1. Multi storey steel buildings constructed for the Olympic games of Athens on 2004 (a) International Television Centre and (b) Main Press Centre

beams are more heavily loaded by vertical loads, compared to the transverse ones, parallel to the above secondary beams. Concrete slabs, the type of which is to be chosen, are supported by the already mentioned secondary beams and the parallel to them main beams. At the end, the system ensuring the lateral stability of the structure is to be specified. There are two main options: either to design all or some of the beam-to-column joints as moment resisting, and therefore have a 3D structure able to resist horizontal loading by frame action, or to arrange, in selected places, vertical bracings, extended over all the height of the building. Instead of steel vertical bracings concrete walls or other compact steel or composite elements could also be used.

At the initial phase of design, the requirements of the architectural design and the electrical-mechanical equipment should be considered. A successful initial arrangement of structural elements is critical for an economic solution, as well as a reliable and efficient structural behavior during the building's life. The application of advanced methods of structural analysis does not correct probable disadvantages of the conceptual design. Oppositely, an advantageous conceptual design leads to a satisfactory structural behavior, despite the application of simple models for structural analysis, as the beneficial behavior, during their lifetime, of buildings in the USA, built in the beginning of the 20^{th} century, showed.

7.2.2 Columns

For the columns of multi storey buildings wide flange I-sections are usually selected, as having a satisfactory strength and stability resistance against flexural buckling, in respect to both main axes of the cross-section. When lateral stability of the building is provided by frame action, or by combined frame and bracing action, crossed double I-sections are many times used (Fig. 7.2), where one section is continuous, while the other is divided in two parts, welded to the continuous one. Another usual option, is to use hollow sections, hot or cold formed, which, if square, have the same rigidity in respect to both axes. The external dimensions of the square hollow sections vary usually between 250 and 400 mm and the thickness between 4 and 30 mm. There is, in this case, the possibility to use for all columns, cross-sections with the same external dimensions and different thickness, depending on the loading level of each column. In the application of hollow columns more elaborated constructional details have to be provided for the beam-to-columns connections. Apart from the aforementioned steel cross-sections, composite steel-concrete cross-sections are very often used for columns, either as hollow sections with concrete infill, or as open steel cross-sections partially or totally encased in concrete (Fig. 7.3). Composite columns exhibit higher resistance and rigidity compared to steel columns, as the concrete is, mostly, under compression and therefore effective. They exhibit also a better ductility, especially under seismic conditions, as the concrete protects steel elements against local buckling phenomena and it is fully encased when inside of hollow sections.

Fig. 7.2. Steel buildings column with crossed double I-section

Composite columns exhibit, in addition, a very satisfactory behavior under fire conditions, their available fire resistance being often sufficient, without protective paints or encasement into isolation boards. In hollow sections with concrete infill, the concrete is reinforced with adequate reinforcing bars. In high temperatures the external steel section loses its strength, but the remaining reinforced concrete core is able to resist existing loading. It is to keep in mind that fire is an accidental load situation, where mainly vertical loads exist, not increased by a partial safety factor, so that the corresponding forces design values are smaller than those included in the fundamental load combination. Instead of reinforcing bars, an I-cross-section could be inserted into the hollow section, before concreting, that is connected to the concrete through shear connectors, at the top and the bottom of the column at each storey. For totally encased into the concrete open steel cross-sections, the increased fire resistance is due to the protection of steel components offered by the concrete, while in the case of partially encased steel sections additional reinforcing bars are needed to obtain the above increased strength.

The procedure for concreting composite members and columns depends on the type of element. For I-sections, partially encased in concrete, where concrete is placed on both sides of the web, between flanges, (Fig. 7.3b), concreting is executed in two steps, with the steel member in horizontal position. First one side is

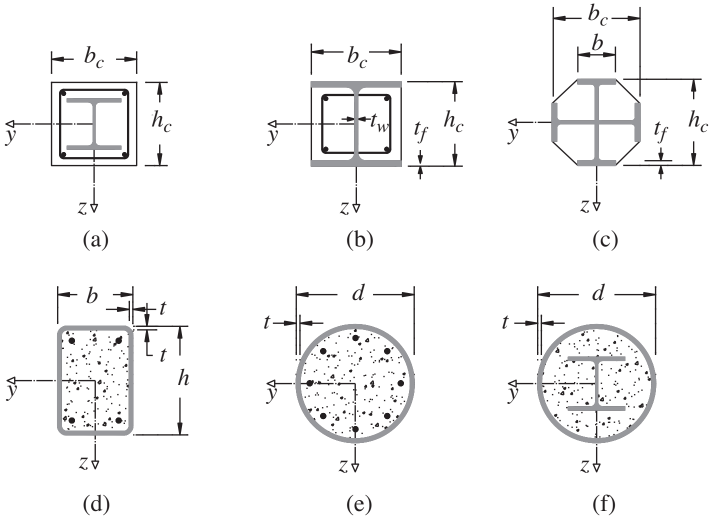


Fig. 7.3. Usual cases of composite columns cross-sections

concreted, and 2 to 3 days later, the second one, after turning the steel member. The areas of joints remain free of concrete in order to allow for bolting and welding operations. Consequently, they are to be protected against fire conditions. For totally encased steel sections (Fig. 7.3f), shuttering is needed. Concreting is executed separately, at each individual storey, starting from the lower level, while in higher stories the erection of the steel structure continues. Care should be taken for an adequate compaction of the concrete.

Infilled concrete in hollow steel cross-sections (Fig. 7.3d, e, f) may be concreted by two different procedures as following: (a) for smaller steel cross-sections, concrete is poured from the top of the column, by successive small parts, of about 30 to 50 cm height, ensuring for each of them a sufficient concrete compaction, by using internal or external vibrators, and (b) for larger cross-sections, concreting may be executed from the lower level of the column to the top, under pressure. To this end, a hole is provided in the wall of the column, near its base, where the muzzle of a pump is connected. The application of this procedure allows, for concreting of a column in more than one stories and a better concrete compaction. At the top of the concreted part a hole should be provided, for the air and overflow escape.

Steel columns in multi storey buildings could be arranged as pinned or fixed at their bases. The arrangement of anchor bolts follows the recommendations presented in chapter 6 for single storey buildings. In principle the columns, in multi storey buildings, are continuous along the height of the building and the beams span between them. However it is possible that the beams remain continuous and the columns are interrupted at floor levels, especially when beams transfer to columns mainly vertical loads and the lateral stability of the building is ensured by vertical bracing systems or shear walls. The continuity of the columns is usually provided, at selected cross-sections, where reduced values of the bending moments exist, us-

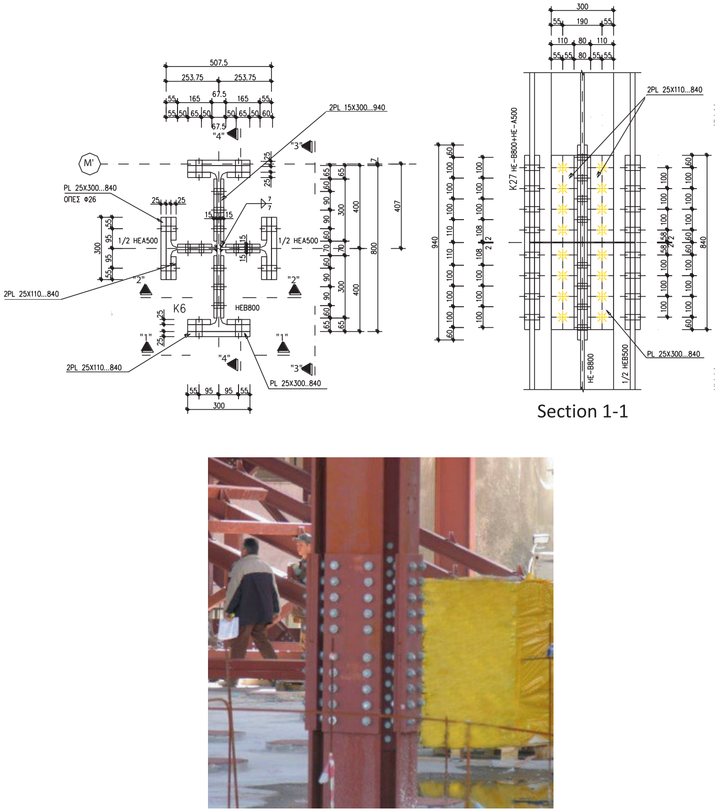


Fig. 7.4. Continuity of a column with crossed I-section. Bolted connection using flange and web plates

ing, mostly, bolted connections with flange and web plates (Fig. 7.4), or through top and base plates, especially for hollow sections. In structural systems without a frame action the continuity of the column could be realized inside the concrete slab. In columns with hollow sections the continuity may also be ensured by full penetration butt welds after an appropriate preparation of the column ends to be welded.

Depending on the architectural design, columns may be placed externally, internally or on the face of the building. When columns are external, they constitute an important architectural element. Such columns are less and differently exposed to fire conditions, compared to internal elements within the building. They are also exposed to different environmental conditions, and they reach different temperatures compared to the internal structural elements, an effect that should be considered in fire analysis. Finally, beams connected to external columns constitute thermal bridges concerning insulation of the building, and therefore specific measures should be taken.

The design of composite structural elements is prescribed in EN 1994/Part 1.1 [7.1]. This code provides design rules for members with usual cross-sections shapes,

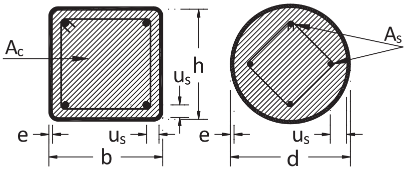
(Fig. 7.3), namely hollow cross-sections infilled with concrete, or open steel cross-sections completely or partially encased into concrete. The above part of EN 1994 gives methods to determine the resistance of composite members against axial compression, and interaction formulae for the simultaneous action of compression and uniaxial or biaxial bending. Rules are also included on important issues such as local buckling of the steel parts, slenderness limitations, required depth of the concrete cover, maximum percentage of the cross-sectional area of the reinforcing bars, compared to the total external area of the column, as well as the influence of the concrete shrinkage. Finally, in the same part, recommendations are provided concerning the fatigue resistance of composite members, as well as the connection between the steel cross-section and the concrete part of the composite columns, in the load introduction area, through shear connectors.

The fire resistance of composite columns subjected to compressive forces without or with some eccentricity is prescribed in Part 1.2 of EN 1994 [7.2]. Detailed Tables give the fire resistance of several types of composite columns, in relation to (a) the external column dimensions, (b) the thickness of the basic steel cross-section members (web, flanges), (c) the level of the axial force, expressed as the ratio n between the acting design force in the fire combination of actions, and the resistance of the cross-section in the normal design situation, (d) the depth of the concrete cover in respect to the main steel cross-section and the reinforcing bars, and (e) the total sectional area of the reinforcement bars as percentage of the overall column cross-section.

As an example Table 7.1 gives the fire resistance of composite columns made of concrete filled in hollow sections. The Table is valid for hollow sections with a wall thickness no more than the 1/25 of the smaller external dimension of the column's cross-section and of a grade of the steel reinforcing bars S 500. For instance let's consider a composite column made from a 260 $\times$ 260 $\times$ 10 steel hollow section, infilled with concrete, which is reinforced with 8 bars of 18 mm diameter, S 500 quality, having a distance of 30 mm from the internal face of the steel wall. From Table 7.1 results that, when the column is subjected to an axial compressive force smaller than the 47% of its compression resistance in normal temperature conditions, it possesses a fire resistance of 60 min (R60), under the standard fire exposure determined in EN 1991-1-2 [7.3], without taking additional protection measures.

The buckling length of columns for a multi-storey frame may be determined through the buckling length coefficient β given by eq. (4.8). This coefficient may be calculated by linear buckling analysis (LBA) of the complete structure, as outlined in section 2.4.1. Another, more practical, method is by means of appropriate charts which were developed by Wood R. H. [7.4] and were included in previous editions of EN 1993-1-1 [7.5]. This method determines the buckling length of a column, in a specific storey, by accounting for the stiffness of the column under consideration, the stories above and below, and the stiffness of the beams adjacent to it at the top and the bottom of the storey, as indicated in Fig. 7.5.

Table 7.1. Minimum cross-sectional dimensions, minimum reinforcement ratios and minimum axis distance of the reinforcing bars of composite columns made of concrete filled hollow sections



steel section: (b/e) ≥ 25 or (d/e) ≥ 25

		Standard Fire Resistance				
		R30	R60	R90	R120	R180
1	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leqslant 0.28$					
1.1	Minimum dimensions h and b or minimum diameter d [mm]	160	200	220	260	400
1.2	Minimum ratio of reinforcement $A_S/(A_C + A_S)$ in (%)	0	1.5	3.0	6.0	6.0
1.3	Minimum axis distance of reinforcing bars u_s [mm]	-	30	40	50	60
2	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leqslant 0.47$					
2.1	Minimum dimensions h and b or minimum diameter d [mm]	260	260	400	450	500
2.2	Minimum ratio of reinforcement $A_S/(A_C + A_S)$ in (%)	0	3.0	6.0	6.0	6.0
2.3	Minimum axis distance of reinforcing bars u_s [mm]	-	30	40	50	60
3	Minimum cross-sectional dimensions for load level $\eta_{fi,t} \leqslant 0.66$					
3.1	Minimum dimensions h and b or minimum diameter d [mm]	260	450	550	-	-
3.2	Minimum ratio of reinforcement $A_S/(A_C + A_S)$ in (%)	3.0	6.0	6.0	-	-
3.3	Minimum axis distance of reinforcing bars u_s [mm]	25	30	40	-	-

The buckling length coefficient β, as above, is calculated through the distribution coefficients n_1 and n_2 corresponding to the head and the bottom of the column (Fig. 7.5):

$$\eta_1 = \frac{K_c + K_1}{K_C + K_1 + K_{11} + K_{12}} \tag{7.1}$$

$$\eta_2 = \frac{K_c + K_2}{K_C + K_1 + K_{21} + K_{22}} \tag{7.2}$$

Where $K = I_C/L_C$ is the column rigidity coefficient (I_C the second moment of area of the column's cross-section), K_1, K_2 are the rigidity coefficients of the same column

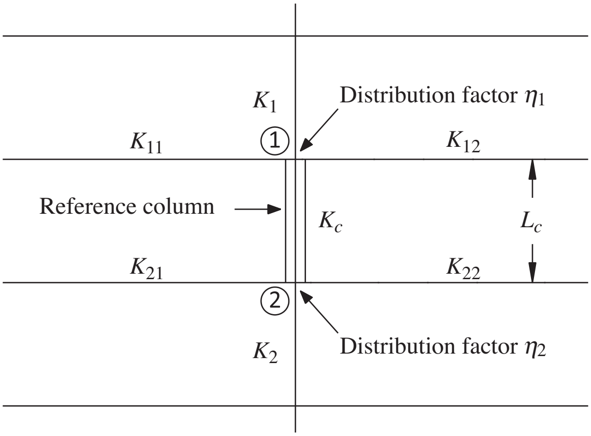


Fig. 7.5. Distribution factors η_1, η_2 for continuous column

in the stories above and below it, and K_{ij} the coefficients for the effective rigidity of the beams connected to the column at its upper and lower end, given in Table 7.2.

Table 7.2. Effective stiffness coefficients for beams in building frames

A. Beams in steel frames		
Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)	
Fixed at far end	1.0 I/L	
Pinned at far end	0.75 I/L	
Rotation as at near end (double curvature)	1.5 I/L	
Rotation equal and opposite to that at near end (single curvature)	0.5 I/L	
General case. Rotation θ_a at near end and θ_b at far end	$(1+0.5\cdot\theta_b/\theta_a)$ I/L	
B. Reduced beam stiffness coefficients due to axial compression		
Fixed	$1.0I/L(1-0.4\cdot N/N_E)$	
Pinned	$0.75I/L(1-1.0\cdot N/N_E)$	
Rotation as at near end (double curvature)	$1.5I/L(1-0.2\cdot N/N_E)$	
Rotation equal and opposite to that at near end (single curvature)	$0.5I/L(1-1.0\cdot N/N_E)$	
In this table $N_E = \pi^2 EI/L^2$		
C. Beams in steel frames with concrete floor slabs		
Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	1.0 I/L	1.0 I/L
Other beams with direct loads	0.75 I/L	1.0 I/L
Beams with end moments only	0.5 I/L	1.5 I/L

The coefficient β is then calculated from the relations:

$$\beta = 0.5 + 0.14(n_1 + n_2) + 0.055(n_1 + n_2)^2 \tag{7.3}$$

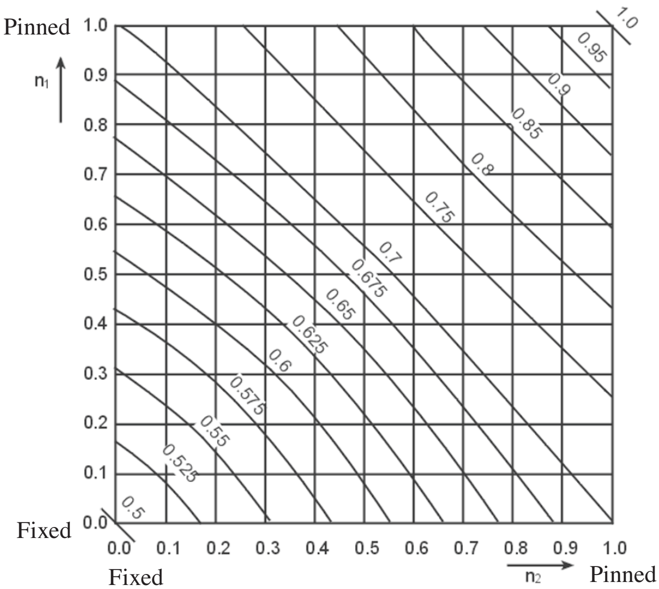


Fig. 7.6. Buckling length coefficient for columns with non-sway ends

for columns having non sway ends, and

$$\beta = \left[\frac{1 - 0.2(n_1 + n_2) - 0.12 n_1 n_2}{1 - 0.8(n_1 + n_2) + 0.6 n_1 n_2} \right]^{0.5} \tag{7.4}$$

for the case of columns with sway ends.

The values obtained from equations (7.3) and (7.4) may be approximately determined using the diagrams of Figures 7.6 and 7.7 respectively. It may be seen that for columns with non-sway ends the coefficient β takes values between 0.5 and 1.0, while for columns with sway ends values greater than 1.0. It should be said that in the method it is assumed that the ratio N_{Ed} / N_{cr} (N_{Ed} the acting axial force and N_{cr} the critical buckling load) has not significant variation in the successive parts of the continuous column. If this is not the case the method leads to conservative results.

7.2.3 Main beams

7.2.3.1 General

The main beams of the building join the column heads at floor levels, in both main directions. The main beam spans are related to the columns' grid and could vary between 6 and 18 m, but in specific buildings the span could be larger. Beams are usually of I-section, hot rolled or built-up. In cases of larger spans, truss beams could also be employed. The webs of main beams may be provided with systematic or isolated openings, to facilitate the installation of the building facilities equipment. Especially for air conditioning ducts, which have often significant dimensions, it should

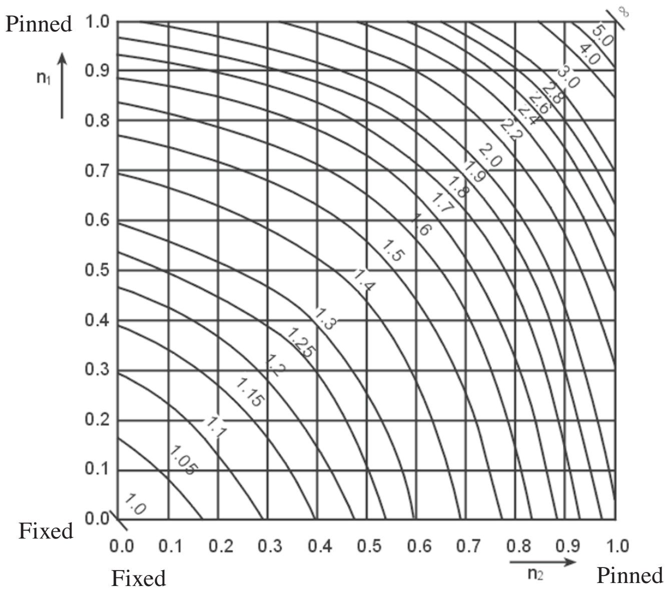


Fig. 7.7. Buckling length coefficient for columns with sway ends

be decided at the beginning of the design whether they travel through the beam webs, or if they'll be in contact with their lower flange, in which case an increased storey's height is needed.

Main beams may either have a composite action in cooperation with the concrete slab or operate as pure steel elements. In both cases, the design cross-section is considered to be pure steel near beam-to-column joints of moment resisting frames, due to the fact that negative moments arise in the joint region, the slab is in tension and consequently not active due to cracking. In buildings where vertical bracing systems ensure the lateral stability, main beams may be arranged as simply supported and are designed as composite for resisting vertical loads. For usual applications, cross-section may be doubly symmetric. For larger spans or higher loads, they may be of mono-symmetric sections with a wider lower and a narrower upper flange of sufficient width to accommodate the arrangement of more than one row of shear connectors, if needed. Typical cross-sections of composite beams are shown in Figure 7.8, corresponding to the usual cases of solid (a, b, c) or composite slabs (d, e, f, see 7.2.5.2), mono-symmetric sections (Fig. 7.8d) or double symmetry cross-sections, with trapezoidal sheets having exaltations parallel (Fig. 7.8e) or perpendicular to the beam (d, f).

Trapezoidal sheets, used for the construction of the composite slabs, span simply between beams, leaving a free space for welding of the shear connectors on the top flange (Fig. 7.8e). When relatively thin sheets are used (according to EN 1994-1-1 with a thickness not more than 1.25 mm), sheets may run continuously over the beams and shear connectors be welded to the beam flange through the metal sheets (Fig. 7.8d, f). For pre-dimensioning of main beams the following indicative values

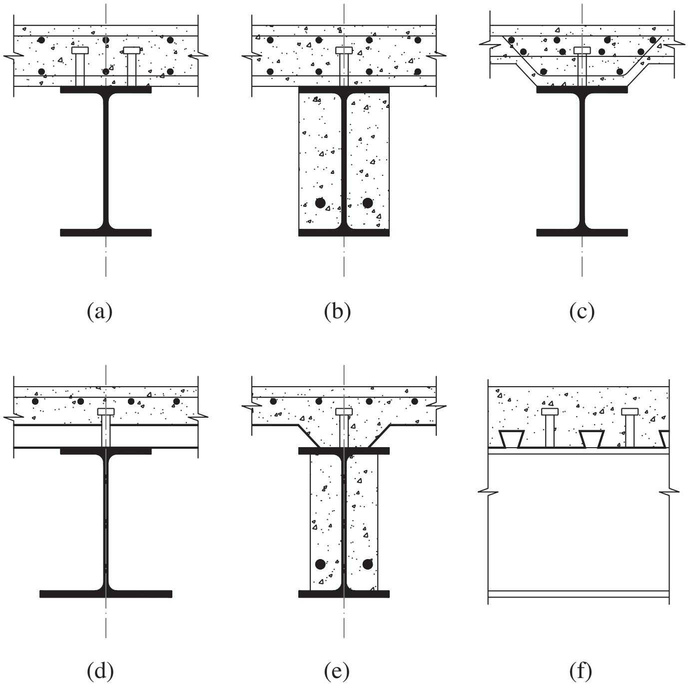


Fig. 7.8. Typical cross-sections of composite beams (EN 1994-1-1)

apply: (a) ratio between span and total depth of the beam (steel beam plus concrete) 15 to 18 for simply supported beams and 18 to 22 for continuous beams, (b) concrete quality C25/30 and (c) shear connectors Φ 19/150 mm. The webs of beams may be protected against fire conditions through encasement in concrete.

Besides cross-section's strength, beams shall also be verified for lateral-torsional buckling (LTB) stability, at the ultimate limit state (ULS), at both construction and service stages. More critical is the construction stage, before hardening of the concrete, where the slab acts as vertical loading and does not cooperate with the steel beam to develop a composite action. On the contrary, at the service state, vertical loads are higher but beams operate as composite, their top flange being restrained against lateral deformations due to the diaphragm action of the slab. Accordingly only the bottom flange could be subjected to LTB deformations. Since this is restrained to twist from the top flange and the web, LTB is seldom critical for the beam at service stage.

Beams shall also be verified against excessive deformations at the serviceability limit state (SLS). Deformations could create problems in the functionality of the building (including the function of probable machinery), water accumulation, damages to the non-structural elements, discomfort to the people, or problems related to the aesthetic of the building as well as to its durability and appearance. The significance of this check becomes more important in contemporary structures, with significant spans and beams from high strength steel.

Deflections shall be checked at both construction and service stages. This verification is often critical, especially for un-propped construction, most usually due to large deflections at the construction stage, where the cross-section is of pure steel and limited rigidity. However, stiffness increases dramatically at the service stage due to composite action, so that deflections are not of concern. To limit deflections, especially at construction stage, propped construction or pre-cambering might be used.

Deformations are always determined by elastic analysis, under the frequent load combination (see section 1.4.3). The maximum acceptable limits of deflections should be defined in the project specification, for each individual project, and agreed with the owner of the building. The National Annexes could specify limit values.

The maximum deflection of a beam can be expressed as:

$$w_{max} = w_1 + w_2 - w_0 \tag{7.5}$$

where w_1 is the deflection due to the permanent loads, w_2 the part of the deflection due to the variable actions and w_0 the pre-camber, if specified.

Usually applied limits for the beam deformations are for the floors L/250 for w_{max} and L/300 for w_2. For non-accessible roofs the values L/200 and L/250 respectively are used. For crane supporting beams more demanding limitations apply (see clause 6.6).

Vibrations in beams could be induced by machines, the synchronized movement of people or ground borne from traffic. In the case of machines it is to arrange for the beams or for a specific part of the building a vibration natural frequency different to the one of the machinery. For excitations due to the users, the natural frequency should be kept above appropriate values. These values are to be determined, as already mentioned, in the project specification and agreed with the client. For usual buildings, the natural frequency of floors should be larger than 3 Hz and for floors where people could move on a synchronized way (dance floors, gymnasium hall, theater scene and similar) larger than 5Hz or 7Hz in more demanding applications. The above values, 3 or 5 Hz, could be considered as ensured if the sum $w_1 + w_2$ is less than 28 mm and correspondingly less than 10 mm.

EN 1994-1-1 provides rules for the verification of composite beams in respect to cross- section verification, resistance against lateral-torsional buckling, resistance of the web against shear buckling and longitudinal shear in the composite cross-sections. The above verifications are extensively presented in chapter 4 of this book.

7.2.3.2 Beams with openings in the webs

Beams with isolated or multiple openings in the webs are often used in buildings in order to facilitate the installation of the buildings equipment (Fig. 7.9). In the usual cases of hexagonal or circular openings, beams are fabricated from rolled profiles, after cutting the originate beam in two parts and reassembling it, as it is shown in Fig. 7.10. In this way the depth and the stiffness of the beams increases. The geometric properties of the openings in the usual case of prefabricated beams with multiple hexagonal openings (castellated beams) are $\phi = 60^\circ, h = 1.5$H, $s_o = 0.5h_o$, $a_o = h_o$,

$c = H/4$ and spacing between centers of openings $p = h$ (Fig. 7.10a). Cellular beams can be fabricated with spacing between centers of circular openings varying from $1.08h_o$ to $1.50h_o$ (h_o the openings diameter). Therefore spacing can be regulated to have full web section in the connections with transverse beams. At the ends of the beams, openings, sometimes, should infilled with appropriate plates to obtain adequate shear resistance, when needed.

The general checks for beams with openings in the webs follow the provisions of EN 1993-1-1. However additional verifications are required that are related to the local conditions at the openings, and more specifically: (a) the local bending of the T-sections (flange and part of the web) above and below the opening's ("Vierendeel" bending), caused by the shear forces across the opening (Fig. 7.11) and (b) the shear and buckling resistance of the part of the web that remains between the edges of closely spaced openings (web-posts).

Fig. 7.9. Castellated beams in steel buildings

The openings may be reinforced by longitudinal and, sometimes, transversal stiffeners (Fig. 7.12a,d) to increase the above specific resistances. Stiffeners could be placed at one or both sides of the web (Fig. 7.12b,c) and should extent at an anchorage distance l_v outside the edge of the opening (Fig. 7.12a), in order to be effective at the opening edges, which are the most stressed areas. The deflection of a beam with openings in the web is greater than the one corresponding to a beam with equivalent solid web section, due to the "Vierendeel" effect. Therefore, the influence of the shear stiffness of the beam should be introduced in the calculations [7.6]. For the usual cases of simply supported beams, with multiple openings in the webs, the ratio between the additional deflection, due to web openings, and the deflection of the corresponding equivalent beam with solid web section, has an indicative value between 0.10 and 0.18. Beams with openings in the webs can also be used in composite structural members. In such case the rules of EN 1994 additionally apply.

EN 1993-1-13 [7.7] is a part of Eurocode 3, under preparation, dealing with beams having openings in the webs. Rules for the additional verifications at the openings are included, for the cases of beams with isolated circular, elongated or rectangular web openings, beams with multiple or closely spaced openings, cellular beams with circular openings and beams with castellated openings. The field of application of the above rules is limited to the cases of no very slender webs ($h_w/t \leqslant 121\varepsilon$, for ε see eq. (3.26)) and for beams with small axial forces $N(N \leqslant N_{pl}/50$, where N_{pl} the plastic axial force of the cross-section, see eq. 3.7). Alternative advanced methods suitable for computer analysis, are also included in this Part. For the deflections of simply supported beams with multiple circular or hexagonal openings under uniformly distributed load, the following approximation formula is given, valid for the usual case where $s_o \geqslant 0.35h_o$:

$$\frac{w_{add}}{w_b} = 3.5n_0 \left(\frac{h_0}{h}\right)^3 \frac{h_0}{s_0} \left(\frac{h}{L}\right)^2 \tag{7.6}$$

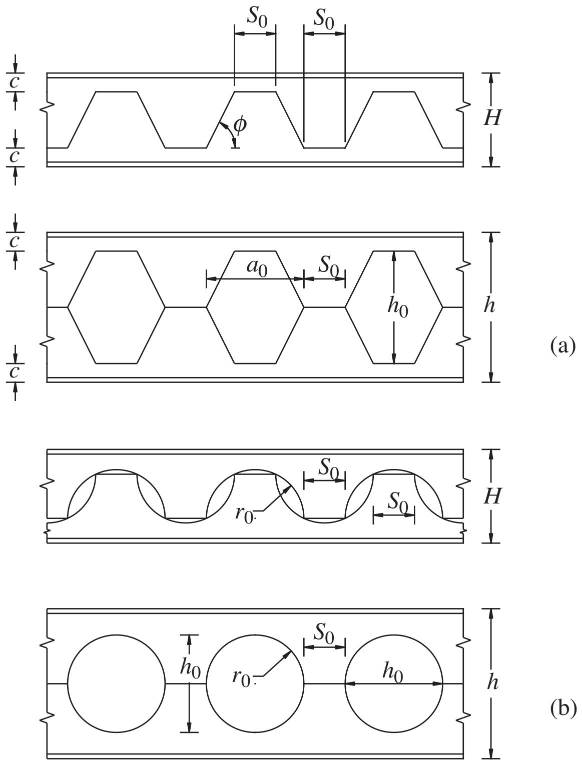


Fig. 7.10. Procedure for the fabrication of beams with multiple openings in the web with (a) hexagonal and (b) circular shape

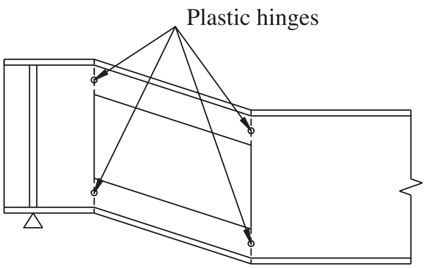


Fig. 7.11. Vierendeel mechanism around a web opening

Where w_{add} is the additional deflection at mid-span, due to the web openings, w_b is the deflection of the equivalent solid web cross-section, n_0 is the number of web openings, L is the span of the beam. For the other symbols see Fig. 7.10.

Concerning constructional tolerances, the dimensions of openings should not exceed the specified ones by more than 1%.

7.2.3.3 Shear connectors in composite beams

For composite beams, main or secondary, shear connectors are welded on the upper flange of the steel beams, ensuring the cooperation between the two materials (steel and concrete). The most usually applied shear connectors' type, is the shear stud with geometric properties, before and after welding, as shown in Fig. 7.13a. The welding procedure is operated with the aid of ceramic ferrules (Fig. 7.13b). After positioning, (Fig. 7.13c), the connector is lightly lifted and an electric arc is briefly struck between the stud and the flange of the beam (Fig. 7.13d). The ceramic ferrule forms a combustion area around the weld location, concentrates the arc in a small region and reduces the heat loss and the cooling rate. In addition it protects the welder from both arc and spatter. After a short welding time, the connector is plunged into the melted steel (Fig. 7.13e) and an annular weld collar is formed around its base (Fig. 7.13f). The ceramic ferrule is used once and is removed after solidification of the molten metal. The specification for arc stud welding operation and qualification is EN ISO 14555 [7.8]. The different types of studs, including shear connectors, their mechanical characteristics and dimensions as well as their designation, are specified in EN ISO 13918 [7.9]. The same Norm provides also characteristics of the ceramic ferrules.

The surface of the beam flange shall be clean before welding. Layers of paints, rust, grease or any type of coatings, should be removed from the weld location, mechanically or chemically. The studs should be fabricated from materials for which the hardness increase after welding is low. For non-alloyed steel studs this requirement is considered to be satisfied when the C-content is less than 0.20%.

The welding equipment consists of the power source, the control unit (which in most systems is combined with the power source), the movable fixture and the welding cables dimensioned so that non-permissible heating is avoided. The welding current in A-(ampere) is arranged at an indicative value of 80d, for studs up to 16

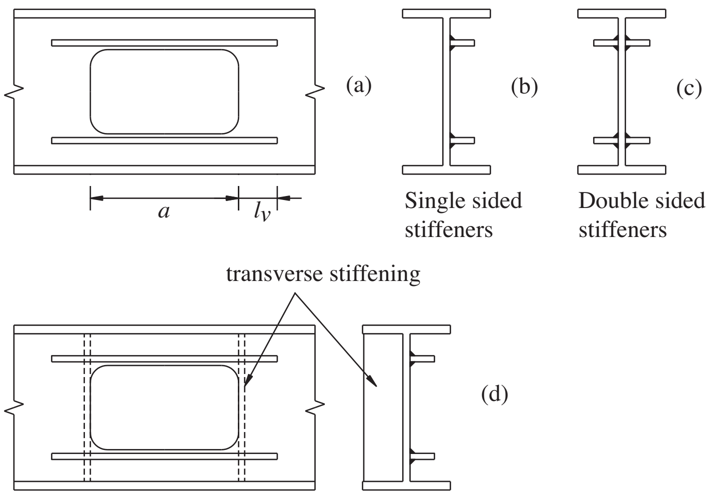


Fig. 7.12. Stiffening of web openings

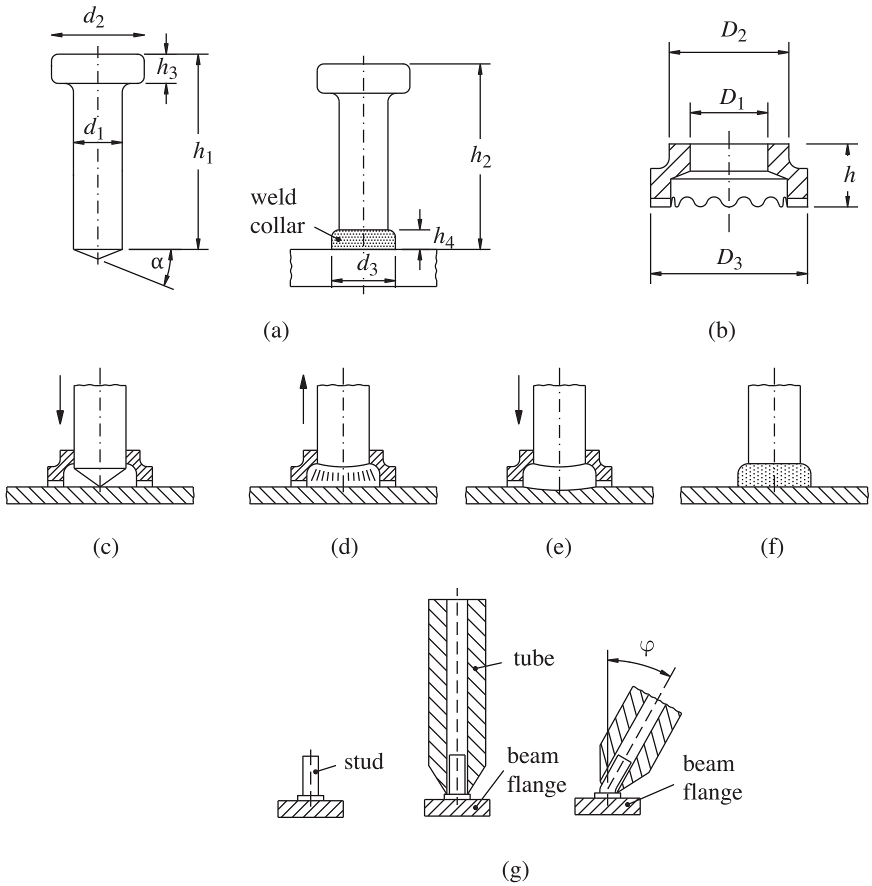


Fig. 7.13. (a) Typical shear connector before and after welding, (b) ceramic ferrule (EN 13918), (c) to (f) Sequences of a stud application, (g) bending test (EN 14555)

mm of diameter (d the stud diameter in mm), and 90d for studs with diameter over 16 mm, while the arc voltage is arranged between 20 and 40 V. The welding time (in sec) is approximately taken equal to 0.02d for studs up to a 12 mm of diameter (d in mm) and to 0.04d for studs over this diameter. The stud lift (Fig. 7.13d) is taken proportional to the stud diameter from 1.5 to 7 mm. Stud welding can be applied in any position.

In general, the flange of the main beams is not covered by the trapezoidal sheets, used for the composite slabs and as shuttering of the concrete. The studs can be welded to the beams in the shop or on-site. Studs may be welded through profiled steel sheeting when their nominal thickness does not exceed 1.5 mm for non-galvanized steel sheets and 1.25 mm for galvanized ones. In the last case the nominal thickness of galvanizing, on each face, should not exceed 30 μm, while the sheets should be maintained in close proximity to the steel beam. Gaps exceeding 2mm cause a high number of defective welds. Through sheets welding is usually applied in secondary beams.

The usual imperfections in the stud welding are related to the quality and shape of the collar and the sensitivity to fracture phenomena. Collar imperfections may be a reduced and irregular form, eccentric position, undercut of the welding surface, reduced height or large lateral projections. The above imperfections can be inspected following a visual examination. The sensitivity to fracture phenomena, such as fracture above collar after sufficient deformation, weld fracture, tearing of the beam's flange or tearing within the weld, are investigated mainly by testing. Fracture is sometimes related to high porosity in the area of the weld. In addition to the above, studs application should avoid damages on the reverse side of the flange. Corrective actions are mainly related to the appropriate adjustment of the welding procedure parameters, such as weld time, current level, lift magnitude, plunging speed, centering of the stud etc.

The required testing for the acceptance of studs welding is specified in EN 14555, which provides the permissible size for each individual imperfection, as well as the permissible total imperfections area. The extent of controls depends on the level of the quality requirements. In usual buildings, where standard quality requirements are applied, according to EN 729-3 [7.10], the total area of imperfections shall not exceed 10% of the stud area. Tests are performed in the shop, to approve the welding procedure specification (WPS), and on-site. In the WPS, all factors influencing the efficiency of the welding should be specified, such as studs' material, compatibility with the structural beam's quality, surface preparation, welding equipment, sequence of welding, application parameters, as well as ceramic ferrules quality.

Tests are visual, destructive and non-destructive. Visual examination is extended to all studs and should cover all imperfections related to the size and the shape uniformity of the collar. The usual non-destructive test is performed by bending of the stud, with the aid of a tube, to an angle of 60^o, as shown in Fig. 7.13g. A weld is considered to pass the test if no cracks are detected in the weld after bending. Bending is applied in the direction of the shear stresses developed in the interface between the two materials. A successfully tested stud remains bended on its place.

The welding personnel shall be approved according to EN 1418 [7.11], while welding coordination should be in accordance with EN 719 [7.12]. In EN 14555, other than bending tests are also described such as tension test, torsion test, radiographies. A shear connectors' verification is presented in the numerical example of composite beam, included in 4.7.3.

7.2.4 Secondary beams

The columns' grid in steel buildings has usually larger dimensions compared to concrete buildings, leading to the need of a concrete slab of significant thickness, if it would span between main beams, and adding an important mass to the overall structure. For this reason, between main beams, secondary beams are provided at a usual spacing between 2.0 and 3.0 m. Trapezoidal sheets span between those secondary beams with their ribs perpendicular to them.

Secondary beams are usually introduced as simply supported beams. They are therefore subjected only to positive bending moments, the concrete is in compression, and the composite action is very efficient, with concrete construction within an effective width of the slab. End supports to main beams are executed as bolted, simple shear connections, transferring only vertical forces. Secondary beams may exist on either one or both sides of the main beams. The simple connection could be constructed through double angles with equal or unequal legs (Fig. 7.14a), through fin plate welded on the web of the main beam, (Fig. 7.14b), that facilitate a connection with an angle between connected parts, different than 90^o, or through end-plates provided on the secondary beams which are bolted to the flange of T-sections, that are welded to the main beams (Fig. 7.14c). In the last support detail cutting of the upper flange of the secondary beam is avoided, but the support force is introduced with some eccentricity, resulting in the transfer of torsional moments to the main

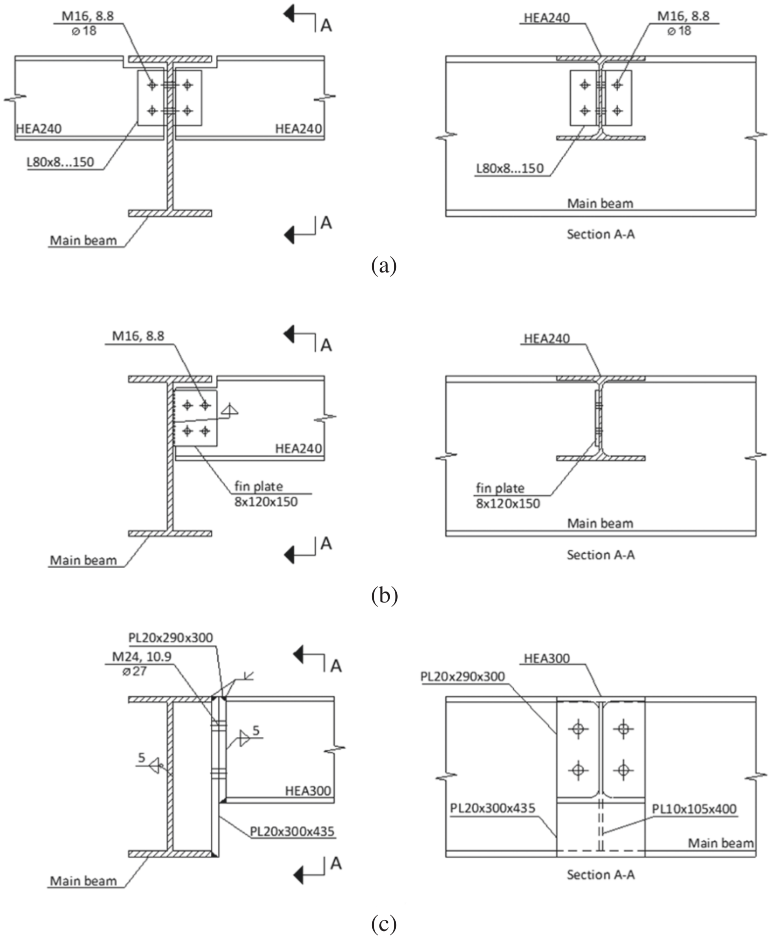


Fig. 7.14. Alternative details for the connection between secondary and main beam

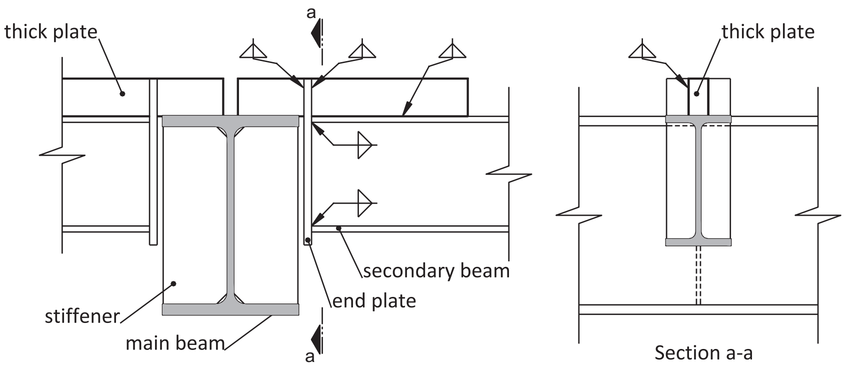


Fig. 7.15. Connection between secondary and main beam through a thick plate welded on the top flange of the secondary beam

beams, which, however, are resisted by bending of the concrete slab. Finally a possible support detail is through a thick plate welded to the top flange of the secondary beam in combination with an end-plate (Fig. 7.15). The extended thick plate is put down on the top flange of the main beam, making a quick erection of the secondary beams possible, avoiding cutting of their upper flange, as well as the introduction of secondary torsional moments.

Most usually the top levels of the secondary and the main beams coincide. Both main and secondary beams could be provided with shear connectors to operate as composite members (Fig. 7.16a). There is also the possibility to place the secondary beams at a lower level, in respect to the main beams (Fig. 7.16b). The slab is put on the secondary beams to cover the difference between the two levels, in which case only the secondary beams are composite. Although the easiest execution, the secondary beams are seldom sitting on the top flange of the main beams, due to the increase of the construction depth.

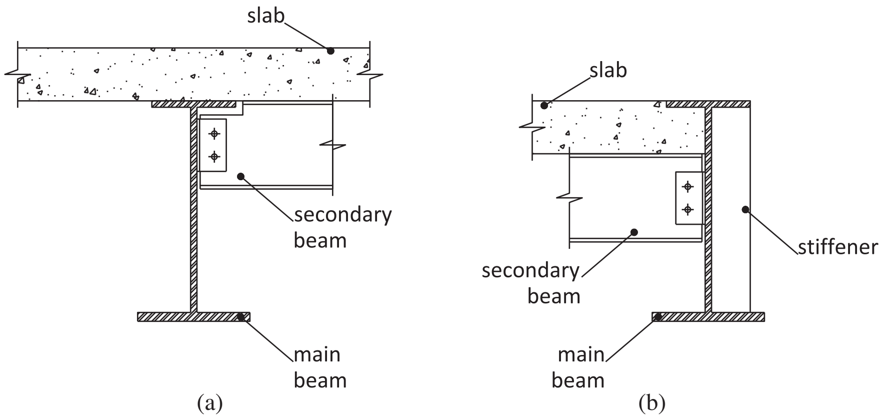


Fig. 7.16. Relative positions between secondary and main beams

For a preliminary design of the secondary beams, the ratio between their span and the total height of the cross-section (steel part plus deck) could be taken between 18 and 20 for simply supported beams and between 22 and 25 for continuous beams, with a shear studs arrangement of Φ 19/150. As for main beams, the deflections of secondary beams should be checked at both the construction and service stages and be propped or pre-cambered as necessary. Secondary beams could also be provided with repeated or isolated openings to their webs, as already mentioned in 7.2.3 for the main beams, or to be constructed as light trusses.

7.2.5 Concrete slabs

7.2.5.1 General

Slabs, in the steel multi storey buildings, have to play two different roles: transfer of vertical loads to the steel beams and working as floor diaphragms, distributing horizontal loads evenly to the columns. Slabs are mostly supported by the top flange of the steel beams. However, there are alternative solutions where the slabs are placed within the depth of the steel beams. In the following possible slab configurations will be discussed.

7.2.5.2 Composite slabs

Composite slabs are the most frequent method of construction for steel multi storey buildings. They consist of two main components: steel trapezoidal sheets and in situ concrete. Steel sheets rest on the steel secondary beams and are used as permanent shuttering to wet concrete. Before concreting light reinforcement grids are installed, with a small cover to the upper face of the slab, to avoid cracking of concrete or, for continuous slabs, to reinforce the slab in the area of negative moments. At the service stage, steel sheets and concrete act compositely, due to the adhesion between the two materials, which is ensured by appropriate small grooves or exaltations in the sheets. The construction method, as discussed above, facilitates a quick execution of the works and therefore it is very often called “fast track construction”.

Trapezoidal steel sheets are industrial products offered in different geometries and thicknesses. Each type of sheet is produced with a standard width, usually between 600 and 900 mm. The thicknesses vary normally between 0.75 and 1.5 mm, the ribs have axial distances b_s from 150 to 300 mm (Fig. 7.17), while the depths $h_{p,}$,for usual application range, vary between 50 and 100 mm. The ribs have a re-entrant trough profile (Fig. 7.17a) or an open trough profile (Fig. 7.17b). Re-entrant trough profiles allow an easy installation of hanging bars without drilling of holes. The total depth h of the slab varies, usually, between 120 and 180 mm. The ratio between the span of the slab and the total slab’s depth, may be taken, in preliminary design calculations, equal to 32 for simply supported slabs or equal to 35 for the end span and to 38 for the intermediate spans of continuous slabs.

The steel sheets can be placed as simply supported elements, between successive steel beams or, more usually, run continuously over more beams. Continuous sheets

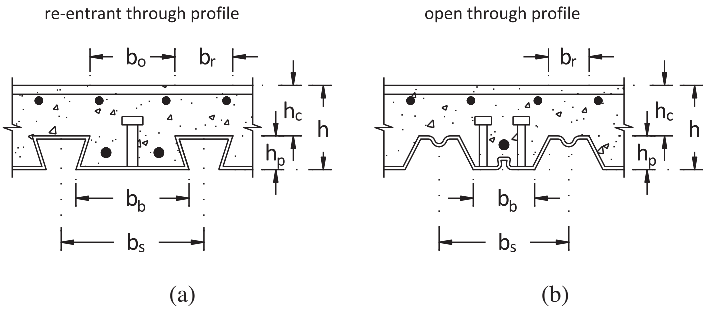


Fig. 7.17. Typical cross-sections of composite slabs

have more advantageous bending moment distribution and smaller deflections, especially in the construction stage, under the wet concrete. However, they require on-site installation of the shear studs to the supporting steel beams, with through sheets welding, an operation not allowed for thick sheets or at indentations. When the deflection of the sheets at the construction stage, during concreting, exceeds the serviceability limitation, propped construction should be used, which is the case for relatively larger sheet spans (3.0 m or more). Sheets' fabricators provide tables with the capacity of composite slabs, including the deflection limitations at construction and service stage, the former being usually more critical. As mentioned before, shear studs should be welded over the flat parts of the sheet and over indentations where they are not in contact with the upper flange of the beam. When indentations exist only in the middle of the sheet flange shear connectors should be installed alternating from one and the other side (Fig. 7.17b).

EN 1994-1-1 gives design rules on composite slabs and more specifically procedures to determine the bending resistance for hogging and sagging moments, where concrete is under compression or correspondingly tension, as well as rules to determine resistance to longitudinal, vertical and punching shear. Information is provided only for slabs spanning in the direction of the ribs and for sheets with narrowly spaced webs ($b_r/b_s \leqslant 0.6$, see Fig. 7.17). If the slab is working compositely with the beams or if it is used as a diaphragm, its minimum total thickness should be 90 mm and the minimum concrete thickness h_c , over the ribs, 50 mm. Otherwise the above minimum dimensions are specified as 80 and 40mm respectively. Transverse and longitudinal reinforcement, usually as reinforcement mats, shall be provided within the depth h_c of the concrete. The minimum reinforcement cross-section in both directions should be 80 mm^2/m. The minimum allowed thickness of steel sheets is 0.7mm. The bearing width at the end supports on steel beams should be not less than 50 mm or 70 mm at intermediate supports. The support can also be indirect through specific parts. The serviceability limit deflection of the steel sheets at the construction stage, due to their own weight and the weight of the wet concrete, is specified to be equal to L/180.

Profiled steel sheeting is designed according to EN 1993-1-3, [7.13]. This part provides rules to examine if the requirements for considering that the slab offers lateral support to the steel beams are fulfilled. Concerning durability, a zinc coating with a total zinc quantity of 275 gr/m^2, including both faces of the sheet, is required for internal floors in a non-aggressive environment, which corresponds to about 20 μ thickness on each face. When one face is visible, an additional protective thickness of 15 μ is required for internal surfaces and 25 μ for external.

Specific measures should be taken when sheets are exposed to particular atmospheric conditions. In general, regarding zinc coating of the sheets, EN ISO 14713 [7.14] applies.

As far as fire conditions are concerned, the composite slab should have, besides the required resistance against the above conditions (resistance criterion), adequate thickness to avoid the danger that fire is spread in the upper storey (insulation criterion). This criterion is considered to be fulfilled when the temperature at the upper surface of the concrete after the critical time, required by the fire resistance class, is less than 140^{O}C. When this temperature limitation is satisfied, it is considered, in terms of strength, that, during fire, the metal sheets lose their strength, while the upper compressive concrete zone can develop its full capacity. For this reason additional reinforcing bars are provided within each rib of the trapezoidal sheet (Fig. 7.17) to replace, in the fire situation, the lost steel of the sheeting and the corresponding capacity to resist tensile forces. It is considered that the reinforcing bar obtains the temperature of the surrounding concrete, so that the distance of the reinforcing bars from the exposed to the fire concrete surface should be specified in the design and indicated in the drawings.

Due to the low thermal conductivity of the concrete, the temperature is slowly propagating into the concrete mass protecting the steel. For instance, in a slab of uniform thickness exposed to the standard fire, after a fire duration of 60 min the concrete temperature is about 640^{O}C at 10 mm from the exposed surface, 420^{O}C at a distance of 30 mm, 250^{O}C at 50 mm and 100^{O}C at a distance of 100 mm. The requirement that no flames or smoke have the possibility to penetrate through the slab (integrity criterion) is considered to be satisfied due to the continuity of the steel sheets.

EN 1994-1-2 deals with the fire resistance of composite slabs and provides a procedure to calculate resistances for both sagging and hogging bending moments, taking into account the variation of the slab thickness due to the shape of the sheets. It is considered that the composite slab can develop, under fire conditions, its plastic strength. The insulation criterion, as mentioned above, is considered to be fulfilled when the minimum concrete thickness (in the area of the sheet ribs) is more than 60mm for a required fire resistance of 60 min and 100mm for a required resistance of 90 min.

7.2.5.3 Slim floors

As an alternative, slabs could be arranged within the depth of the steel beams, in order to reduce the total thickness of the system. To this end steel beams, from hot rolled or built-up cross-sections, are provided with a wider lower flange, compared to the upper one, to allow an easy support of the trapezoidal metal sheets. These sheets are deeper than those used for the simple composite slabs, as discussed before, with a depth of about 200 mm, as there are not secondary beams and sheets span between the main beams (Fig. 7.18). In addition to the advantage of a reduced deck thickness, and therefore of a reduced storey height, slim floors provide a better fire resistance, as the main steel beams are partially encased in the concrete. As an alternative, instead of composite slabs, prefabricated concrete slabs could be introduced between main steel beams. Many types of slim floors are offered by different producers, having steel beams of several profiles with, in some cases, a systematic holing at webs of hollow sections to facilitate the flow of the concrete inside the section.

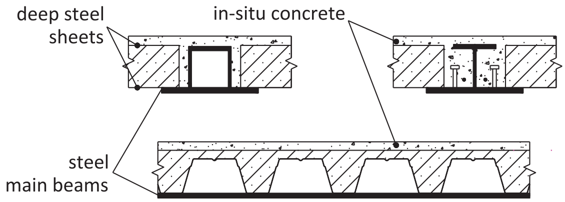


Fig. 7.18. Types of slim floors

7.2.5.4 Concrete slabs

Instead of composite slabs, reinforced or pre-stressed concrete slabs could also be used applying alternative procedures: (a) in the place of metal sheets prefabricated thin concrete slabs are used as shuttering, on which in situ wet concrete is added, (b) totally prefabricated concrete slabs, compact or with voids. In the axes of the main steel beams an empty space is left to allow the introduction of shear connectors, which space is finally filled by a no shrinkage mortar and (c) traditional solid concrete slabs, which very rarely apply.

7.3 Beam to column joints

7.3.1 Introduction

Beam to column connections and joints could be classified as: (a) simple, able to resist only forces, and having sufficient rotation capacity to be modeled as pinned, (b) rigid, able to resist both forces and moments, and having sufficient rigidity to consider that the angle between the connected members remains unchanged during loading and (c) semi-rigid, in which a change ϕ of the angle between connected members appears, when a moment M applies on the connection, which cannot be neglected, and the relation M-ϕ should be introduced in the analysis.

There is a large variety of connection types. In the following the most usual types will be presented. It is reminded that it is important to provide a correct representation of connections and joints in the analysis model, by introducing pins, springs or rigid elements as appropriate. Most commonly, beam to column connections are bolted connections since beams and columns are linear elements transported on-site, where they are connected during assembly.

Joints are characterized by the shape of their M-ϕ curve. In practice there are no ideal rigid or ideal pinned joints. Chapter 5 (section 5.5.5) presents the criteria and the procedure to classify joints according to their strength and their rigidity, and gives practical rules concerning the limits within which simplifications in modelling the joints could be used. It is to clarify that as "connection" usually is characterized the interface between connected parts and their connecting means (bolts, welds) while as "joint" the overall area of the connection in which, in addition, the end part of the beam and the part of the column between beam flanges (column flanges, web panel, stiffening elements) are included. In the above classifications the strength and the rigidity of the joint are considered.

7.3.2 Simple connections

The transmission of shear forces is usually realized by bolting the web of the beam to the column, through a plate (fin plate) welded perpendicularly to the column flange or web (Fig. 7.19, case a). Instead of plates equal or unequal leg angles could be used, welded to one of the connection members and bolted to the other. As an alternative, bolting of the beam could be done through a welded end-plate of the beam

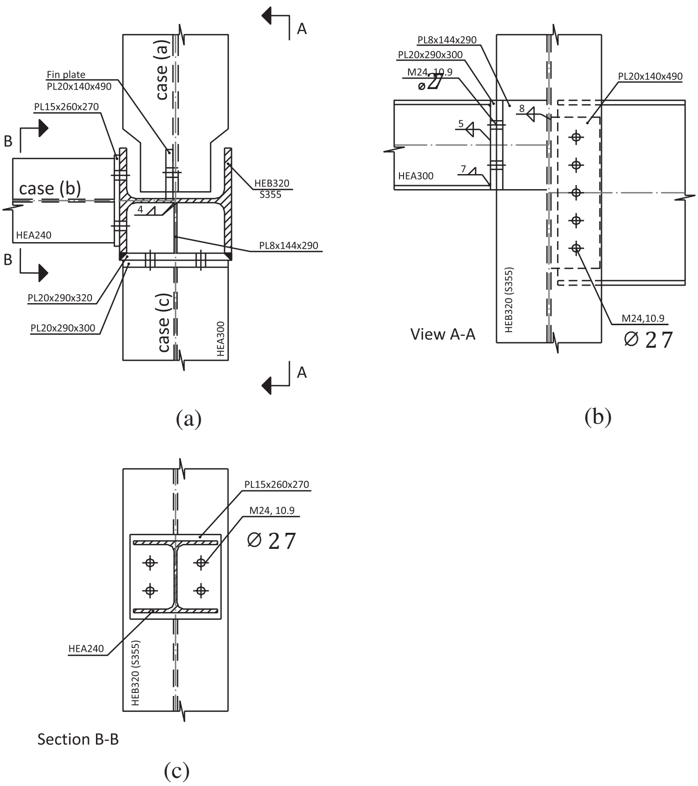


Fig. 7.19. Simple beam to column connections

(Fig. 7.19, case b) using bolts near the main axis of the beam's cross-section. In the transverse direction the connection can also be realized at the ends of the column's flanges through auxiliary plates or part of an H-section (Fig. 7.19, case c). This eccentric arrangement facilitates bolting when beams from both main directions are connected to the column at the same node. Figure 7.20 shows an example of simple connection of a beam to a hollow section column. Instead of a plate, an I-section is welded to the column, as it is stiffer and stronger to lateral forces, so that finally the connection is a bolted one between two I-shaped beams. When simple connections are used throughout the beam-to-column joints of a multi storey building, lateral stability should be ensured by introduction of vertical bracing systems.

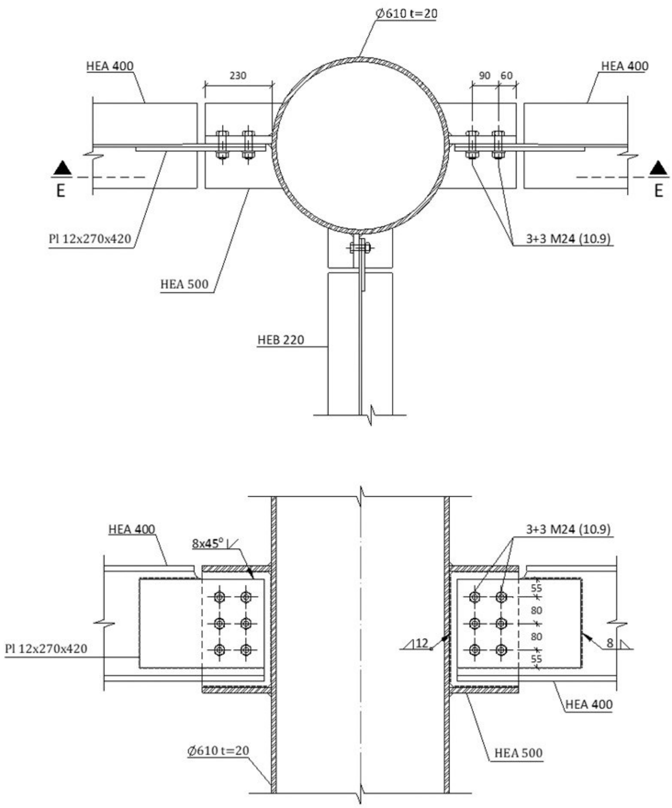


Fig. 7.20. Simple connection of a beam to a column of a circular hollow section

7.3.3 Rigid connections

A usual arrangement for rigid beam-to-column joints is to weld at the beam an end-plate, and bolt it subsequently to the column flange. The end-plate is usually extended beyond the top flange of the beam to increase the lever arms, by providing

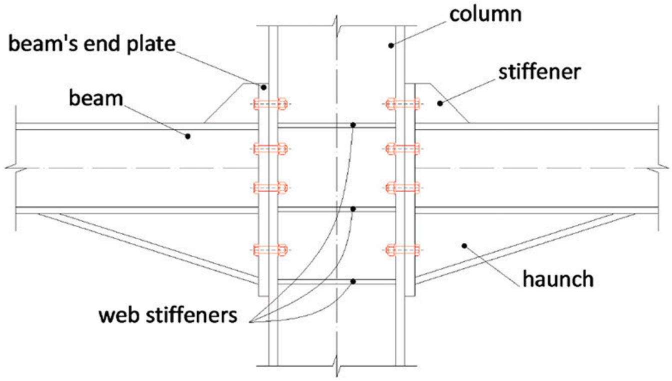


Fig. 7.21. Rigid beam to column connection through a beam end plate and a haunch

additional bolts. When the resulting height of the end-plate is not sufficient to provide the required strength and rigidity, a haunch is welded to the beam (Fig. 7.21). To avoid excessive thickness of the above plate, stiffener could be arranged.

The formation of plastic hinges at the interface between the connected parts should be avoided, especially under cyclic loading, as inelastic behavior at this position relies on inelastic elongation of bolts which is usually limited. When a plastic hinge is expected to develop at the joint, a haunch moves the plastic hinge at a distance from the above interface, where the beam has its basic cross-section. In addition the haunch gives the possibility to design a joint with sufficient overstrength against the beam and ensure the position of the plastic hinge. The column web should be provided with stiffeners at positions where concentrated forces are transferred, such as the levels of the beam flanges and the haunch.

An alternative is to weld, in the workshop, short beam sections to the columns and bolt the beams to these sections through web and flange plates (Fig. 7.22). A typical beam-to- column joint, mainly applied in USA, is shown in Fig. 7.23. The beam flanges are welded to the columns on-site, while the web is bolted to a fin plate

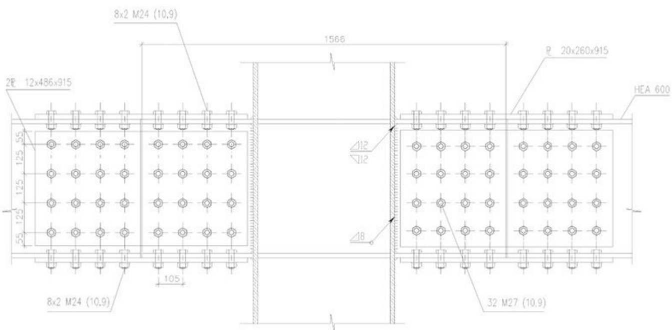


Fig. 7.22. Rigid beam to column connection through short cantilevers welded to the column with flanges and web plates

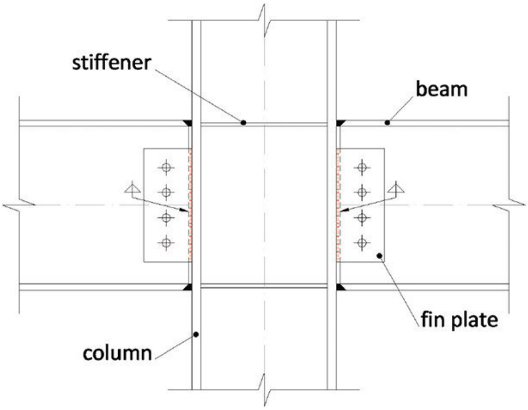
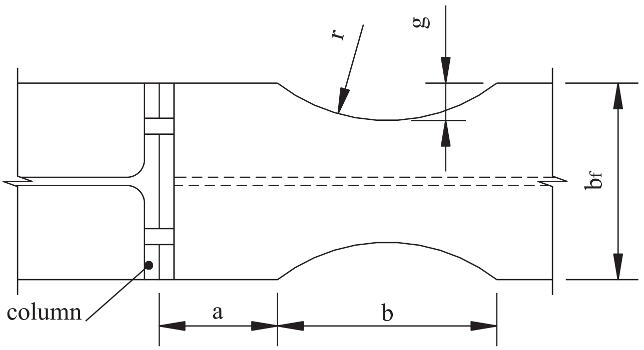


Fig. 7.23. Type of a rigid beam to column connection applied in existing buildings

that is welded to the column in the workshop. This type of connection facilitates erection, however it showed a non-satisfactory behavior under seismic conditions [7.15].

To avoid plastic hinge formation in the area of the connection and lead it to the beam, its cross-section could be weakened at a distance from the connection to achieve a "reduced beam section" (RBS). This weakening could be done by cutting partially the flanges, as illustrated in Figure 7.24. Due to this weakening's shape it is also called a "dogbone" RBS. The cutting g will be specified by the designer and determine the maximum bending moment developed within the RBS and, consequently, the maximum moment at the connection, at the end of the beam. It is recommended that the weakening has a circular shape and that g should not be larger than 25% of the flange width [7.16, 7.17]. For the distances a and b the values indicated in Fig. 7.24 are recommended [7.16, 7.18]. Instead of the flanges' cuts, as above, for the local weakening of the beam, drilled holes could be provided in both top and bottom flanges of the beam.



$a = 0.5$ to $0.75b_f$

$b = 0.5$ to $0.75d_b$

(d_b depth of the beam)

$$r = \frac{b^2 + 4g^2}{8g}$$

Fig. 7.24. "Dogbone" weakening of a beam

In rigid connections on hollow section columns, diaphragms should be provided to avoid local damage on the relatively thin column walls. Diaphragms could be arranged internal to the column which is divided in three parts (Fig. 7.25a), between, above and below the diaphragms. Beam flanges are welded to the diaphragms, while the web is connected to the column by a bolted connection. As an alternative, a short I-section could be fully welded to the column at the workshop, and be connected by bolting to the beam (Fig. 7.25b). In another case of internal diaphragms, the column is divided in two parts, the interruption is made in the middle between beam flanges to allow diaphragms welding (Fig. 7.25c). In the case of external diaphragms the column remain continuous (Fig. 7.25d). A stress concentration is presented at points A [7.18].

When beam-to-column joints are rigid, frames behave as moment resisting for both vertical and horizontal loading. Frames are stable through frame action, without need of vertical bracing systems.

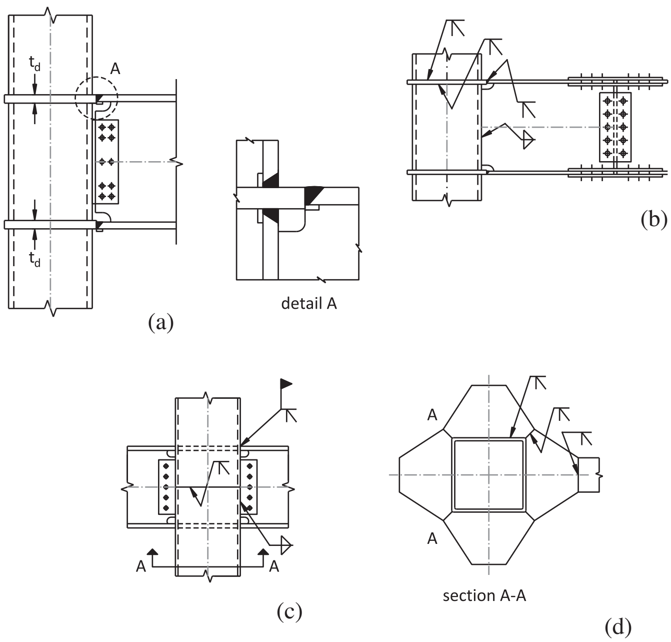


Fig. 7.25. Diaphragms in beam to column joints with a hollow column cross-section

7.3.4 Semi-rigid connections

Semi rigid connections behave between simple and rigid ones. They are able to resist bending moments but the change of angle ϕ, from the initial angle between connected members, when moments apply, may be significant in the measure that influence the bending moments distribution in the frame, to an extent that cannot be neglected in analysis. The connection is characterized by a moment-rotation curve, M-ϕ, that should be introduced in global analysis, most usually in the form of appropriate springs.

In practice, semi-rigid connections are used in structures where horizontal loading is mainly resisted by vertical bracing systems and a weak frame action exists so that connections participate to this end as secondary, auxiliary elements. A usual type of a semi-rigid connection is shown in Fig. 7.26, where both flanges and the web of the beam are connected to the column through angle-sections.

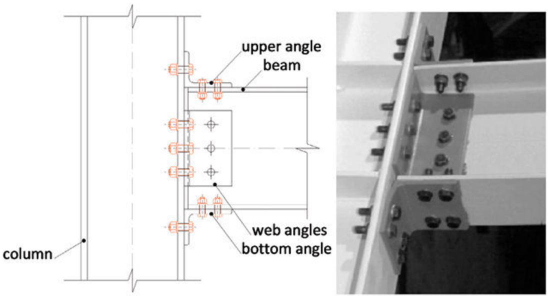


Fig. 7.26. Type of a semi-rigid beam to column joint

7.4 Systems ensuring the lateral stability of the building

7.4.1 Introduction

The main function of the system slabs-beams-columns is to transfer vertical loads to the foundation. However, the structure should be able to transfer also horizontal loading to the ground, which results from wind, seismic action or as well constructional imperfections, such as deviations of columns from verticality.

In concrete buildings, lateral stability is mainly ensured by the monolithic nature of concrete structures and therefore by the possibility of the beam to columns joints to resist moments, i.e. through the development of frame action, as well as through provision of shear walls. In steel buildings there are two possibilities to provide lateral stability and resist horizontal forces: (a) through moment resisting frames that require rigid connections between beams and columns and (b) through vertical bracing systems combined probably with shear walls. In the second option there is the possibility to provide simple beam-to-column connections, which transfer only forces, but no moments. It is possible to provide frame action or vertical bracings in both main directions of the building, or frame action in one and vertical bracing in the other direction, or arrange in one direction simultaneously moment frames and vertical bracings. In the last arrangement beam-to-column joints might have semi-rigid behavior.

Vertical bracings should be preferably arranged symmetrical, along the perimeter of the building, in order to avoid eccentricities and increase the torsional rigidity of the building. However, architectural and functional requirements limit sometimes this possibility. For instance staircases and lifts, where bracings or walls could be located, are usually placed close to the center of the building. In addition, in buildings having main and secondary faces, different architectural possibilities to place bracings exist.

For an efficient arrangement of vertical bracings, the following general criteria apply: (a) arrangement in such a way to give the possibility to resist horizontal actions in both main directions of the building, (b) in each direction at least a pair of bracings should preferably be placed, to share horizontal actions among them. When only one vertical bracing may be located in one direction, i.e. in one side of the building only, there is an eccentricity between the bracing and the geometrical or the mass center of the building, where the resultant of wind or seismic forces apply. This effect results in the development of a torsional action, (c) the rigidities of the two bracings of the same pair should be similar to avoid torsional deformations, (d) the best position for the bracings is in the perimeter of the building, as already mentioned, (e) the bracings should preferably be continuous along the height of the building and (f) the rigidity of this building should be preferably similar in the two main directions of the building.

The efficiency of the vertical bracings arrangement may be checked by means of vibration modes of the building. An efficient arrangement should have: (a) lower modes, having small natural periods, of translational type and (b) concentration of an important part of the building effective mass into the above lower modes. In addition, it should be avoided: (a) torsional lower modes, (b) important diaphragm deformations at floor levels, in the case of flexible diaphragms, (c) corner points of the building with significant differential deformations, such as in cases of L or T-shape buildings and (d) a large number of modes with very small participating mass, that indicates oscillation of isolated members or of small group of members.

In the following conventional types of bracing systems are presented. However, innovative types with exchangeable dissipative parts or damping systems have been developed for use in seismic regions to which reference is made in the literature [7.19, 7.20, 7.21, 7.22, 7.23, 7.24].

7.4.2 Moment resisting frames

As already mentioned, moment resisting frames require the arrangement of semi-rigid or, in seismic regions, rigid beam-to-column joints. Moment resisting connections may be arranged in all beam-to-column joints in both main directions, producing a 3D-frame. Horizontal forces are accordingly resisted by frame action. The view of an axis of such a building frame is shown in Fig. 7.27. Similar plane frames are formed in all axes, as well as in both directions.

A 3D moment resisting frame has a large margin of possibilities to redistribute forces and to develop post elastic behavior due to its high degree of static redundancy. The columns should have adequate rigidity in both directions, so that hollow sections or crossed double I-sections apply.

As an alternative, moment resisting frames can be arranged only in the perimeter of the building, while all internal frames are provided with simple beam-to-column

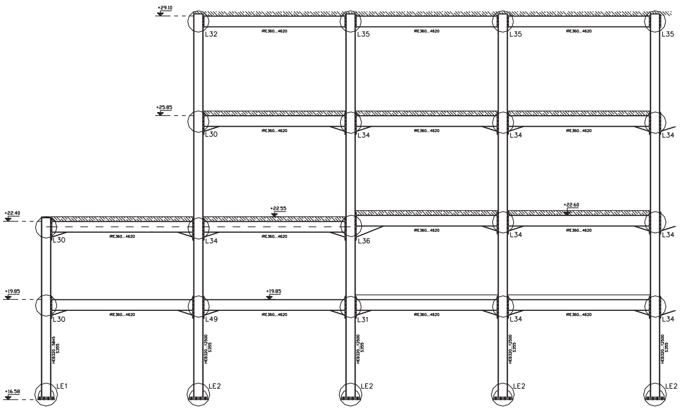


Fig. 7.27. Moment resisting plane frame according to an axis of a multi storey building

connections. The lateral stability of such a building is ensured exclusively by the perimeter frames while horizontal actions are transferred to them through the diaphragm action of the slabs. The columns of perimeter frames may be of I-section and be arranged with their web parallel to the perimeter line of the building. Only columns in the corners of the building require significant rigidity in both principal axes, when they participate in two perpendicular moment frames. On the contrary, internal columns resist only vertical loads and may be H-shaped. They may be continuous along the height of the building, offering additional strength and rigidity in advanced stages of horizontal loading.

Limits of the horizontal displacements under serviceability load combinations are not specified by the Eurocodes, and related limitations are to be specified in the project specification for each individual project or in the National Annexes. On the contrary limitations for the horizontal deformations, including horizontal drifts, are provided for seismic load combinations as outlined in section 7.5. In flexible structures, such as moment resisting frames of significant height, it should be examined whether vibrations caused by dynamic wind actions should be considered.

7.4.3 Concentric bracings

Alternative types of concentric vertical bracings are shown in Figure 7.28. Horizontal forces are resisted mainly through the development of axial forces in the members of the bracing. In the analysis it is usually considered that only diagonals under tension participate in the resistance of horizontal forces. The general remarks outlined in section 6.4.3 for the seismic design of the single storey buildings are also valid, while additional rules related to multi storey buildings are included in 7.5.3. Concerning seismic design, the tensile diagonals are the dissipative members of the structure and measures are taken to ensure that the yielding of the tensile diagonals develops before the failure of the connections at the ends of the diagonals, as well as before the yielding or bucking of the beams and columns. Examples of buildings with concentric bracings are shown in Fig. 7.29.

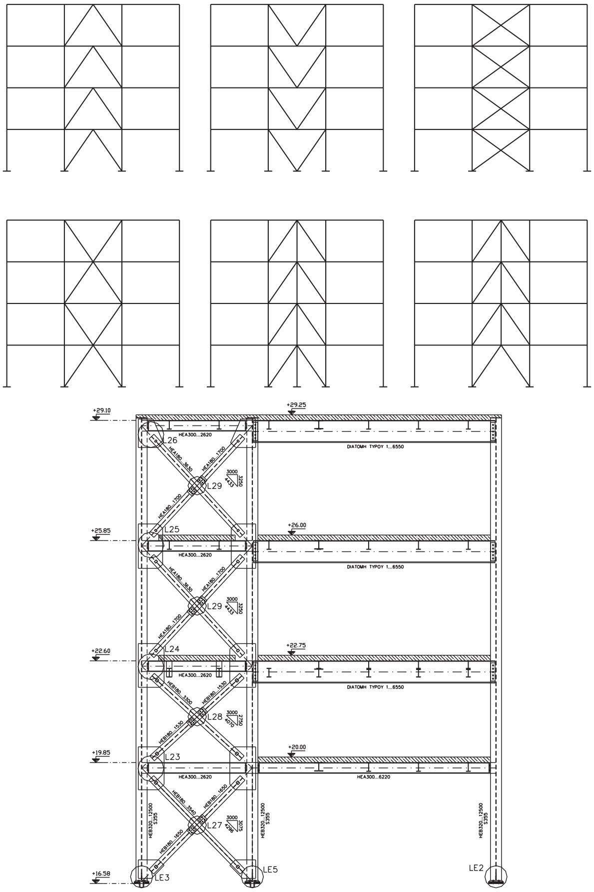


Fig. 7.28. Types of concentric bracing systems

Fig. 7.29. Examples of buildings with concentric bracing systems

7.4.4 Eccentric bracings

Alternative indicative types of eccentric bracings are shown in Figure 7.30. An example of a multi storey building with eccentric bracings is shown in Fig. 7.31. Horizontal forces are resisted by development of axial forces in some members of the bracing and of bending moments in some others. The general remarks presented in section 6.4.3 for seismic design are also valid, while additional rules related to multi storey buildings are included in section 7.5.4. Concerning seismic design, the links are the dissipative members of the system, which can develop plastic deformations due to direct stresses under bending or shear stresses under shear forces, or a combination of both, depending on the length of the link. The ability of the link to develop plastic deformations, avoiding local instability phenomena, is enhanced by adequate web stiffeners. The other members of the bracing are designed with adequate overstrength against the links.

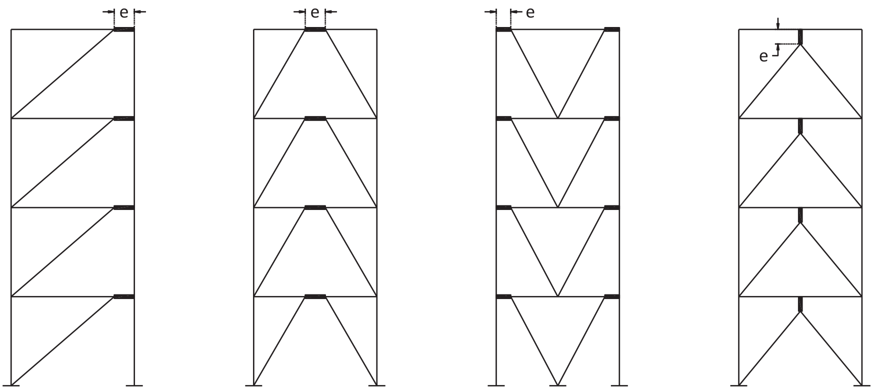


Fig. 7.30. Types of eccentric bracing systems

The connections between beams and columns of the bracing are usually simple connections when the link does not end at a column. In the latter case a moment connection is arranged. The connections of the diagonals to the beam are usually simple connections. They could also be arranged as moment resisting connections. In such a case bending moments develop in the diagonals while their buckling length is reduced for buckling in the plane of the bracing.

Fig. 7.31. Steel building with eccentric bracings

7.4.5 Shear walls

Shear walls composed of steel, concrete or composite steel-concrete panels may be introduced to provide lateral stability to a building. Their arrangement follows the same rules already presented for vertical bracing systems. Concrete cores in the area of stairs and elevators contribute also in the resistance to horizontal loading. When lateral stability is ensured exclusively by shear walls the steel framing is leaning against the walls and resists only vertical loads. Steel beams could be connected to the concrete elements through end plates and anchor bolts. A simplified plane steel structural system with a concrete wall is shown in Fig. 7.32. In Fig. 7.33 an example of a steel building erected around a concrete core is presented.

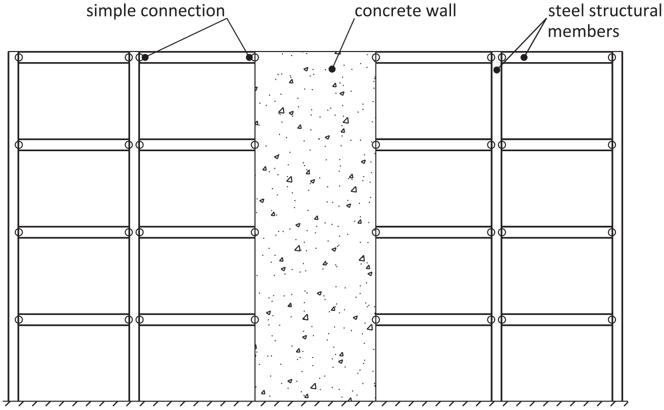


Fig. 7.32. Steel structural system with a concrete wall

Instead of concrete walls, shear walls composed of steel panels have been used in many applications, mainly in USA and Japan. Ductility of the wall may be increased by using specific steels with low yield strength and high deformation capacity. The stress-strain relation of such a steel, with low yield stress (LYS), is from the onset of loading non-linear, without a clear yielding level. Indicative values for such low yield steel are 86 MPa for the, as conventionally defined, yield strength, 250 MPa for the ultimate strength and 50% for the ultimate strain. The provision of transverse and longitudinal stiffeners improves the buckling strength of the wall panels. Steel shear panels, are fastened to a steel frame, composed of horizontal and vertical elements from

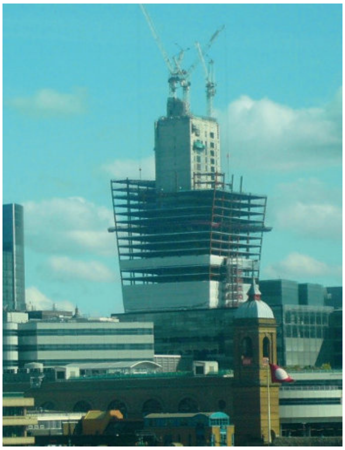


Fig. 7.33. Steel building with a concrete core, under erection

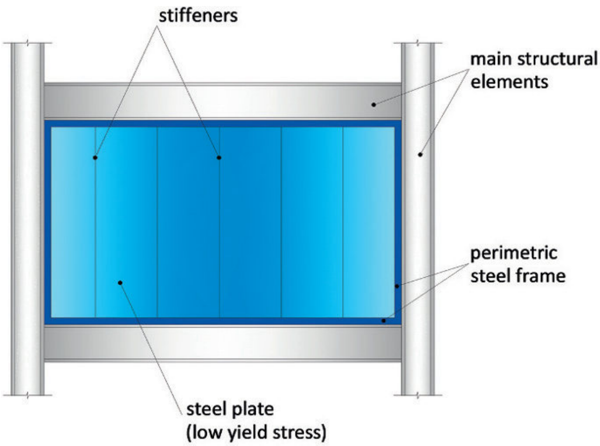
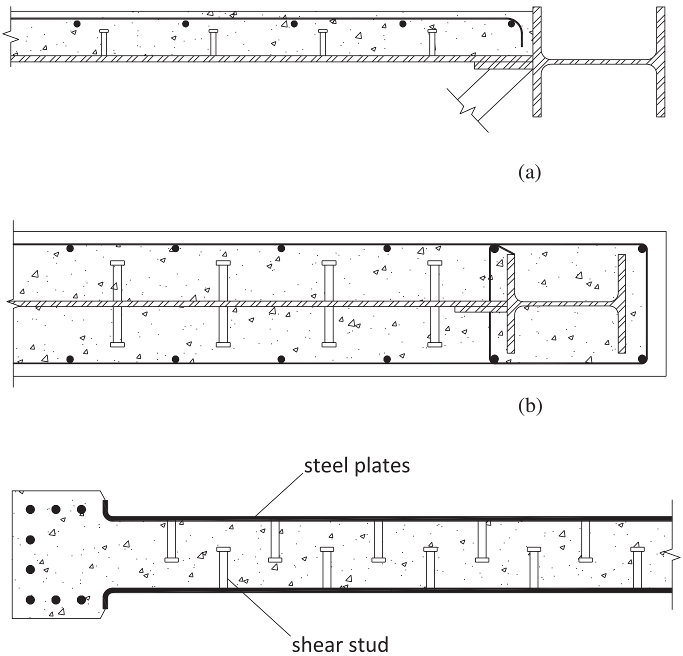


Fig. 7.34. Steel shear wall

I-sections, (Fig. 7.34), to facilitate connection to the rest of the structural elements (beams and columns). An example of a simple building with steel shear walls is shown in Fig. 7.35.

Fig. 7.35. Example of a steel building with steel shear panels

Finally, composite shear walls may be employed. They are composed of one steel plate, partially from one side or completely encased in concrete (Fig. 7.36a, b) or of two external steel plates with an intermediate concrete core, as a sandwich construction (Fig. 7.36c). In all cases the cooperation between the two materials is ensured through shear connectors, welded to the steel plates. Concrete offers to the steel plates a protection against shear buckling and therefore stiffeners are not needed. The end of the walls are formed as columns, by encasement of a steel section (Fig. 7.36b) or by widening the wall and provision of longitudinal and transverse reinforcement, to act as a flange of the wall and provide resistance against overturning moments (Fig. 7.36c). Composite shear walls have been successfully applied mainly in Japan, and showed a very satisfactory behavior during the Kobe earthquake. The rigidity panels of the last two categories, working as infills in the main structural elements, could be applied in new constructions, as well as in the strengthening of existing buildings.



(a)

(b)

(c)

Fig. 7.36. Steel shear wall

7.5 Seismic design to Eurocode 8

7.5.1 General

The general recommendations and rules of Eurocode 8 (EN 1998) [7.25] were presented in section 6.4, with reference to single storey buildings. These general rules are also valid for multi storey buildings for which, in the following, additional, specific to them, rules will be presented.

7.5.2 Moment resisting frames

For applications in seismic areas, structural elements and the structure as a whole should possess sufficient ductility to be able to develop plastic deformations, through which a part of the energy introduced in the structure, during a seismic event, is dissipated. For moment resisting frames it is intended that, during a strong earthquake, a reliable plastic mechanism, through formation of an adequate number of plastic hinges, develops. Before the formation of such a mechanism, any type of failure in a member or a connection should be avoided. To this end, structural members or connections, adjacent to positions of possible plastic hinges, should be provided with sufficient over-strength.

According to EN 1998, moment resisting frames should be designed such that plastic hinges are formed in the beams or in the beam to column connections and not in the columns. This requirement could not be respected at column bases and at the top storey of multi storey buildings. It is evident that the formation of a soft storey plastic mechanism should also be avoided. Following EN 1998, plastic mechanisms in the connections are possible only when special connections are introduced and their capability to develop plastic deformations is studied experimentally and analytically. Otherwise, when the usual types of connections are employed, it should be ensured that plastic hinges develop only in the beams.

The beams should have sufficient resistance against lateral-torsional buckling, assuming a plastic hinge at its most loaded end, under the seismic design situation. In these end cross-sections of plastic hinges it is to be ensured that the plastic resistance and the rotation capacity are not limited by axial compression or shear forces. To this end, for cross-sections of classes 1 and 2, the design axial force, due to the seismic design load combination, should be less than the 15% of the corresponding plastic axial design resistance, while the design shear force less than the 50% of the corresponding cross-section plastic resistance. If the axial force is beyond the aforementioned limit, the provisions of EN 1993, concerning the influence of the axial force on the plastic moment capacity of the cross-section, have to be applied.

According to the strong columns-weak beams approach, it is expected that plastic hinges will form at the beams, where dissipative regions are located. Columns should be verified for the capacity values of forces and moments calculated according to expressions similar to (6.2), presented in section 6.6.4. In that expressions, Ω is the minimum value between the different ratios M_{pl}/M_{Ed} resulting for all beams in which a plastic hinge is probable to form (M_{pl} is the plastic bending moment and M_{Ed} the design value of the bending moment in the seismic design situation). The shear resistance of the web panel zone should also be checked.

Beam to columns connections should be checked following capacity design criteria, to possess sufficient over-strength against the beams. At plastic hinges it is required that cross-sections develop the full plastic strength. In addition it is required that the joints have sufficient rotation capacity, to offer the possibility of a moment redistribution, during the formation of the successive plastic hinges, and of the development of the final full plastic mechanism, without the appearance of local buckling phenomena. The rotation capacity θ_p of the plastic hinge region is defined by

$$\theta_p = \delta/(0.5L) \tag{7.7}$$

as it is shown in Fig. 7.37, where δ is the beam's deflection at mid-span, and L the beam's span. The rotation capacity, as above, should not be less than 35 mrad for structures of a DCH (high) ductility class (see 6.4) and not less than 25 mrad for a DCM (medium) class.

Under the design seismic combination it is required that no collapse of any structural member occurs (no collapse requirement for rare earthquake). The design peak ground acceleration, as determined in EN 1998, has a 10% probability of exceedance in a period of 50 years, in other terms, the peak ground acceleration has a return period of 475 years. National Annexes could specify different values for the above periods and probabilities. Members are checked in the seismic action combinations using the resistances given in EN 1993 for the usual non seismic situations. In addition, it is required that under a seismic action, with a larger probability of exceedance,

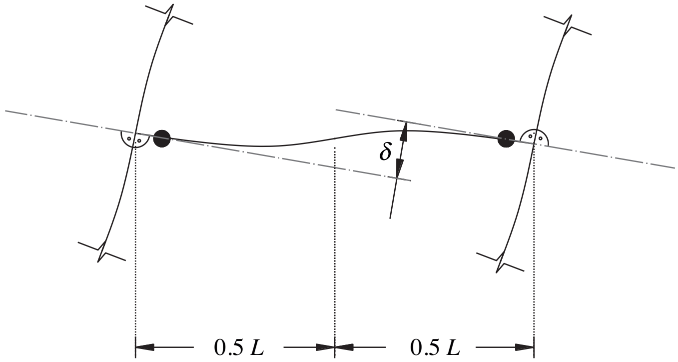


Fig. 7.37. Rotation capacity of a beam in cross-sections where a plastic hinge is developed (EN 1998)

corresponding to a smaller return period, damages to the non-structural elements are limited (damage limitation requirement for frequent earthquake). The recommended by EN 1998 return period for this earthquake is 95 years, but National Annexes could specify other values.

The damage limitation criterion is considered to be fulfilled if the inter-storey drift, i.e. the difference between horizontal displacements at the top and the bottom of each floor, do not exceed, in each direction, a critical value D. More specifically, it is to be verified that:

$$d_r \cdot v \leq D \tag{7.8}$$

where d_r is the design inter-storey drift evaluated as the difference of the mass centers lateral displacement at the top and the bottom of the storey under consideration, v is a reduction factor which takes into account the lower return period of the seismic action, associated with the damage limitation requirement, and D is the maximum acceptable relative displacement.

The design inter-storey drift d_r , as above, is the displacement corresponding to a totally elastic response of the structure, therefore the value calculated for the seismic loading must be multiplied by the behavior factor q introduced in the analysis. The recommended values for the coefficient v, that expresses the ratio of accelerations between the frequent and the rare earthquakes, are 0.40 for buildings of importance classes 3 and 4 and 0.50 for buildings classified in importance classes 1 or 2. It should be mentioned that buildings to EN 1998, are distinguished in four importance classes. Ordinary buildings, for instance, are classified in class 2, others such as schools, assembly halls or other buildings of gathering of people in class 3. The recommended values of the displacement D are as following:

a) 0.005h for buildings having non-structural elements of brittle materials attached to the structure (h is the height of the storey under consideration),
b) 0.0075h for buildings having ductile non-structural elements attached to the structure and
c) 0.010h for buildings without non-structural elements or fixed in a way to not interfere with structural deformations.

The influence of probable torsion of the building on the value of the drift should be considered in the design.

7.5.3 Concentric bracings

The requirements of EN 1998, presented in section 6.4.4 for concentric bracings in single storey buildings, are also valid for multi storey buildings with the following additional remarks: (a) in structures up to two storeys, the limitations for the diagonals' slenderness ratio do not apply and (b) to obtain an homogeneous dissipative behavior of the diagonals, at all levels, it should be verified that the maximum value of the overstrength factor Ω, defined in equation (6.2) does not differ from the minimum one by more than 25%, i.e. $\Omega_{\max}/\Omega_{\min} \leqslant 1.25$.

EN 1998 allows dissipative elements to be in connections between members of the structure. As already mentioned, dissipative connections are under development, while, in practice, structural members are still the only dissipative elements. An example of a connection able to dissipate energy is the INERD type connection [7.19], shown in Fig. 7.38, where energy dissipation is related to bending of a rectangular pin. This connection, which was extensively studied in European Universities, presents important advantages, as pins in bending are highly dissipative elements and easily replaceable parts. The q-factor of the system is of the same order of magnitude as the one for moment resisting frames, while all compression and tension diagonals are active since compression is avoided through the limitation of the compression forces due to yielding of the pins [7.20].

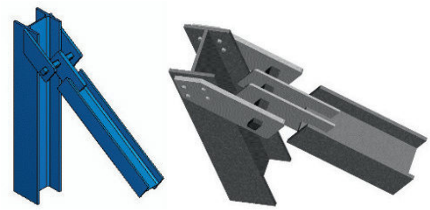


Fig. 7.38. Rectangular pin as dissipative member in the diagonals connection of a concentric X-bracing

The lateral stability of a building may be ensured, in the same direction, using both moment resisting frames and concentric bracing systems. In these cases a unique value for the behavior factor q could be used. Horizontal forces are distributed between the two resisting systems according to their stiffness.

7.5.4 Eccentric bracings

The requirements of EN 1998, presented in 6.4.4 for eccentric bracings in single storey buildings, are also valid for multi storey buildings with the following additional remark:

To obtain a homogeneous dissipation behavior for all links at the different levels, similar values for the overstrength factor Ω should be applied. More specifically, the individual values of the ratios Ω in each level should not exceed by more than 25% a specific value. This value is defined as the minimum between following values: (a) the minimum value of $\Omega = 1.5V_{pl}/V_{Ed}$ among all short links (as defined in 6.4.4) and (b) the minimum value of $\Omega = 1.5M_{pl}/M_{Ed}$ among all long and intermediate links, where V_{pl} and M_{pl} are the shear and bending plastic resistances of an individual link and V_{Ed}, M_{Ed} the design values of the corresponding shear forces and bending moments related to the seismic design situation. For the capacity design of all members of the eccentric bracing, not containing a seismic link, a relationship similar to (6.2) is applied (see section 6.4.4.3), where for Ω the above minimum value is used.

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8
Fabrication and erection

Abstract. In this chapter the procedures concerning the construction of the structure of steel buildings are presented, divided in the in-shop fabrication activities and the on-site erection. Specific sections refer to each production phase, such as cutting and holing, welding, bolting, corrosion protection and erection, as well as to the quality control and the constructional imperfections. In all sections the guidelines and application rules provided in the frame of Euronorms are presented and discussed.

As far as welding is concerned, the relevant sections include: the general rules to be respected during welding procedures, the appropriate preparation of the structural elements to be welded, the usual types of welds' defects, the methods of non-destructive inspection and testing and the limits of acceptable imperfections, in relation to the nature and the importance of the building.

Concerning the surface protection of the steel members, the environment types in relation to their corrosivity, the surfaces preparation, the different paint and protection systems, the methods of inspection and the constructional measures to ensure paints execution, inspection and maintenance, are commented.

A section is dedicated to the quality control and the procedures to be respected during the whole construction, as well as each specific phase, in order to ensure a qualitative, reliable and durable structure. Finally, a specific section is related to the limits of acceptable constructional imperfections within which strength, stability and functionality of the structure are ensured.

8.1 Introduction. Execution classes

An efficient and successful construction of the structural part of a building, which leads to a satisfactory behavior during its lifetime, presupposes an attentive and successful design as well as a diligent and well-organized execution of the works that respects the relevant rules, specifications and quality requirements. To this end the steelwork fabricator should have the experience in similar constructions and should possess the appropriate installations and equipment. The personnel, responsible for the various parts of the works, should have the related experience and the required qualifications. The inspection of the executed works and the corresponding corrections, if necessary, are also an important part of the execution program.

The main parties involved in the construction of a building, are the owner, the authorities, the designer, the contractor and the supervisor. The functions, duties and

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responsibilities of the parties are defined by the body of lows and by their mutual agreements. The steelwork is manufactured by a steel fabricator who may be the contractor or a subcontractor.

All materials to be used (structural steel products, connecting means, welding consumables) should be provided with the necessary certificates, indicating their origin and their quality, as described by the relevant specifications. The materials should be stored, before use, and protected against environmental conditions in appropriate installations. The material certificates will be included, after the end of the works, in the execution documentation.

The execution works are divided into the in-shop and the on-site activities. The in-shop production steps include: cleaning of the structural steel surfaces, usually through sand blasting, application in most cases of a thin protective shop primer, cutting of the steel sections to the desired lengths, shaping of some members (usually curving under hot or cold conditions), holing, end preparation of members, where required, for the execution of the welds, preassembly of linear members to create structural parts which will be transported to the site as a whole, execution of the welds and application of the protective surface treatment. During the in-shop production a systematic inspection of the above works is to be carried out. As a next step the steel parts will be transported to the site and erected in their final place.

The European Norm EN 1090: "Execution of steel and aluminum structures" and especially its part 2: "Technical requirements for steel structures" [8.1] is related to the execution works and includes recommendations and application rules for the quality verification of the steel structures. The extent and the strictness of the quality requirements depend on the importance and the nature of the building. To this end EN 1090-2 provides, in an Annex (of informative character), a classification of steel structures in four categories, called "execution classes", with increasing quality level demands. Each structure, or parts of it, is to be classified in one of the above categories.

The execution classes, denoted as EXC1, EXC2, EXC3 and EXC4, correspond to increasing strictness requirements from EXC1 to EXC4. The class to be applied is related to the whole structure or only to a part of it, to specific structural elements or to specific type of works. The choice of the execution class is decided by the owner and the designer, during the design phase, considering the criteria included in EN 1090 and presented below. The contractor and the project manager, if they are known in advance, should be accordingly consulted. In case where an execution class is not specified in the execution specification, EXC2 is to be applied for buildings.

To classify each project in an execution class the nature and the use of the building, the consequences of a failure, the character of the loading in which the structure is mainly subjected as well as the constructional characteristics are to be taken into account. More specifically three different criteria are considered:

a) The consequence class. The buildings are classified in three classes, CC1, CC2 and CC3. In CC3 class are classified buildings associated with high consequences in case of failure (danger for human life, social, economic or environmental consequences), as for instance public buildings, theaters, power stations etc. In CC2 class, buildings with considerable but not very great consequences are included, such as residential and office buildings and in CC1 class, structures with low

Table 8.1. Structures classification in execution classes (EN 1090-2)

Consequence classes		CC1		CC2		CC3	
Service categories		SC1	SC2	SC1	SC2	SC1	SC2
Production categories	PC1	EXC1	EXC2	EXC2	EXC3	EXC3	EXC3
	PC2	EXC2	EXC2	EXC2	EXC3	EXC3	EXC4

consequences are classified, as for instance agricultural buildings which are not very often visited by people.

b) The service category. Two categories are distinguished: In SC1 category are included structures or components designed for quasi static actions only, structures designed for seismic actions in low seismicity regions and in the low ductility class, (see chapter 6), or structures designed for fatigue actions induced by cranes of low intensity (cranes classified in fatigue class S0, see clause 6.6). In SC2 category are classified structures or components designed for seismic actions in regions of medium or high seismicity, following the rules for medium and high ductility class, (see chapter 6), or structures and components designed for fatigue actions (loading by cranes of S1 to S9 fatigue classes) or structures subjected to vibrations induced by wind actions, rotating machinery etc.
c) The production category. Two categories are also distinguished: In PC1 category are classified non-welded components constructed from any steel grade or components manufactured from steel of grade below S355. In PC2 category are classified components manufactured from a steel grade S355 or higher, components essential for the structural integrity and assembled by welding executed on-site, components with a hot forming manufacturing or components in lattice girders from circular hollow sections requiring end profile cuts.

After classification of the whole structure, or of the individual structural components, following the above three different criteria, the selection of the corresponding execution class results from the matrix given in EN 1090-2 and presented in Table 8.1. Using, as an example, this Table, one can see that a structure classified in CC2, SC2 and PC2 subclasses shall be fabricated following the requirements corresponding to the third execution class (EXC3). It is already mentioned that parts or components of the same structure could be classified in different execution classes.

National specifications could provide complete definitions and criteria for the selection, by case and type of structure, of the appropriate execution class. As example, an indicative, simplified classification is given in Table 8.2. Execution class is also a design issue, since it may be used by the designers to determine controls required during fabrication, in order to meet their specific design assumptions. The owner, in all cases, may also specify a higher class.

It is evident that the fabricated and erected structure as well as each of its components and connections, have usually differences (more or less important) from the ideal geometry. When the existing deviations are sufficiently small, without introducing significant secondary stress conditions, or producing reduced resistances, or creating serviceability problems, they could be accepted. In EN 1090-2 the limits of acceptance, called “tolerances”, are given in many Tables covering all usual constructional cases. The tolerances are distinguished in (a) essential tolerances, corre-

Table 8.2. Examples for execution classes

Execution class	EXC1	EXC2	EXC3	EXC4
Description	Structural components up to S275.	Structures made of steel up to S700.	Structures made of steel up to S700.	Structures and components with extreme consequences for people and/or the environment
Structures	• Low-rise buildings (e.g. 1 to 2 floors) • Agricultural buildings	Multi-story buildings (e.g. <13 floors)	• High rise buildings (e.g. >12 floors) • Pedestrian, road and rail bridges • Crane tracks	• Rail and road bridges of significant importance • Industrial plants with hazardous potential • Safety tanks • Nuclear power plants
Components	Beams low span (e.g. <5m)	Beams middle span	Beams long span	
Details	Fillet welds	Butt welds	Butt welds of thick plates	

sponding to limits within mechanical resistances and stability verifications continue essentially to be valid and (b) functional tolerances required to meet functions, other than resistances and stability, as for example fit up of components or appearance of the structure or other serviceability and functionality problems.

Steelwork contractors are certified to fabricate steelwork up to a certain execution class. For example, contractors with an execution class 3 certification may fabricate structures of execution class 1, 2 or 3 only. The certification is given when the contractor follows a certain factory production control (FPC) system. The manufactured products are then receiving a CE marking that allows them to be legally distributed in the markets of EU states. Contractors may furthermore have an EN ISO 9001 certification concerning the application of a management system for design, manufacture and installation. This certification is however different and should not be mixed with the one for execution class.

8.2 Cutting, holing and shaping

8.2.1 Introduction

Cutting and holing of the steel sections and plates are the first activities related to the fabrication of the steelwork. Steel sections are usually available in the typically produced lengths by the mills. However, if required time, quantities and additional cost are within acceptable limits, a fabricator could order sections having exactly the lengths provided by the design. In such a case, loss of material as well as cutting activities are reduced. In the past, a significant part of the overall work was performed manually (marking of the cutting lines, marking of the holes' centers, operation of the equipment). Nowadays cutting and holing are made automatically, using CNC (Computerized Numerical Control) equipment (see Fig.8.1), which is connected to the electronic drawings of the execution design. In this way the productivity is increased while the possibility of errors decreases.

8.2.2 Cutting

The cutting methods used in the production of steel buildings are sawing, shearing, disc cutting, water jet techniques and thermal cutting. The steel members in their free edges, after cutting, are hardened and their ductility is decreased. Cutting should be performed following procedures appropriate to maintain geometrical tolerances and maximum hardness of the above free edges within the limits specified by the applied standards.

The quality of the cut surfaces is defined according to the criteria (perpendicularity, angularity, regularity) and the classification in ranges specified in EN ISO 9013 [8.2]. EN 1090-2 recommends the range which is to be respected, depending on the execution class of the project. In addition, in order to maintain a minimum ductility as well weldability, for an efficient execution of the welding during the next steps of the production line, the hardness value, after cutting, should remain less than a maximum acceptable value. In EN 1090-2 the above limit is defined as 380 or 450 (Brinell values) depending on the steel quality. The efficiency of the applied cutting process is to be verified by performing appropriate tests.

Depending on the thickness and the desired quality of the cutting surfaces, preheating is possibly needed. Cold cutting is applied for relatively thin members. Laser cutting is used for greater thicknesses of about 20 mm. Plasma cutting or water jet methods are applied for thick elements (100-150 mm). In any case burrs or protrusions preventing the alignment or bedding of the steel elements should be removed.

Fig. 8.1. General view of a steel fabrication workshop

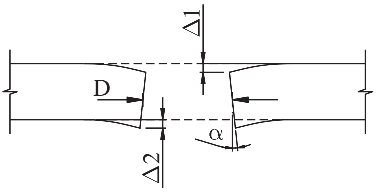
8.2.3 Holing

The methods applied for holing are drilling, punching, laser, plasma or other thermal methods. As already mentioned in 8.2.2, the area near the holes, after holing, is hardened, limiting locally the ductility of the member. All holing methods should leave a finished hole in which the limitations already mentioned for cutting are fulfilled. The final holes' diameters, in relation to the diameter of the bolts to be installed, are already presented in chapter 5.

Punching is in general permitted provided that the thickness of the element is not greater than the bolt diameter or, in the case of elongated holes, the minimum hole's dimension. In projects classified in execution classes 1 or 2, punching without reaming could be applied. For execution classes 3 or 4 punching without reaming is not permitted, the hole should be initially opened and, at a second step, enlarged by 2 mm.

For punched and plasma holing, limits to the distortions presented in the perimeter of the hole are included in EN 1090-2, as it is shown in Figure 8.2. In all cases burrs should be removed before assembly. In cases where holes are opened in one operation, through parts which will remain packed and will not separate after holing, it is necessary that burrs are removed only from the external surfaces. In the position of splices, surfaces which will be assembled together in contact, should be punched in one direction for all components.

Holes for fit bolts could be either drilled directly in full size or reamed in situ. In the last case they should be initially made by 3 mm at least undersized. If the fastener is to be fit through multiple components, they should be held together during drilling or probable reaming. Elongated holes could be punched, as a first step, in one operation, or formed after punching or drilling of two different holes and, in a second step, completed by a hand thermal cutting.



$$D = \frac{d_{max} + d_{min}}{2}$$

$$\max(\Delta_1 \text{ or } \Delta_2) \leq \max(D/10; 1 \text{ mm})$$

$$a \leq 4° \text{ (i.e. 7\%)}$$

Fig. 8.2. Permitted distortions of punched holes and plasma cuts

In EN 1090-2, an imperfection of 2 mm, concerning the deviation of an individual hole from its position within a group of holes, or the deviation of an entire group of holes from its intended position, is considered as an essential manufacturing tolerance (see also 8.1 and 8.7). There is no tolerance in the distance between an individual hole and a cut end. Concerning the holes' diameter, taken as the average of the entry and exit diameters, the tolerance is 0.5 mm (by more or less of the nominal diameter). The above limitations are considered as essential manufacturing tolerances.

Unless prohibited by the project specification, drifts may be used for the alignment of holes. Elongation of holes, for bolts transmitting loads, should remain within the limits of the acceptable imperfections and more specifically: class 1 tolerance for structures of execution classes 1 or 2, and class 2 tolerance for structures of execution classes 3 or 4 (see 8.7). Correction of the misalignment by reaming is preferred.

8.2.4 Shaping

Shaping is a secondary, supplementary activity in the fabrication plan, not existing in all projects and related to specific structural members. It is applied by bending, pressing or forging, to give a desired shape to linear or plane steel elements, usually to curve linear members or to produce cross-sectional shapes starting from plane steel sheets. It could be performed following hot or cold procedures. In all cases it should be executed following specific standards and specifications adapted to each specific steel product, appropriate to maintain, after forming, the mechanical and other characteristics of the initial material. Members which, after shaping, present cracks, lamellar tearing or other damages (for instance to the surface protection) should be rejected.

Hot shaping is not permitted in all steel qualities. For the usual steels up to and including quality S 355, the hot forming process takes place in the red-hot state (temperatures between 400° and 800°C). The temperature level, the timing and the cooling rate should be appropriate for the particular type of steel. Temperature elevation is combined with a simultaneously applied pressure, to conduct the steel member into the desired shape. Such a procedure is used to give a camber, when needed, to beams or purlins. Bending or forging in the blue heat range (250° to 380°C) is not permitted.

Cold forming is usually applied to sheets with a small thickness. It could be carried out by pressing, folding or roll forming. Cold forming leads also to a local hardening of the material and to reduction of the ductility. To this end, in some cases, a stress relief treatment is applied.

8.3 Welding

8.3.1 Introduction

As already mentioned (chapter 5), welding and mechanical fastening are the two main alternatives to connect steel structural parts. In principle, connections executed in the shop are welded connections, while mechanical fasteners are used for connections on-site. However there are cases where welding is also used for on-site connections. As presented in 5.4, during welding, the components' ends to be joined are melted together at a high temperature, produced by an electric arc. The arc is formed, by a power source, between the parent metals and the electrode, which leaves at the joint area additional material, producing, at the same time, protective gases.

The most usual welding processes in buildings construction (see also section 5.4) are the manual metal arc welding, the metal acting gas welding (MAG), the submerged arc welding (SAW) and the tungsten inert gas welding (TIG). For specific cases there are also other welding processes that could be used. In general, the different processes that potentially could be used, are defined in EN 4063 [8.3].

The welding procedures to be applied should be qualified depending on the execution class of the whole steelwork, the parent metal quality and the degree of mechanization. For steelwork of execution classes 3 and 4, especially when high strength steel is used, it could be specified that production tests are necessary to be carried out, before production starts.

Welding is a relatively complex fabrication operation, many technical parameters are introduced in the manufacturing procedure and the personnel to be involved, in all phases and operational levels, must be experienced and appropriately qualified. The welder's qualification should be certified by an authority or an accredited body. Details about welder's qualification are specified in EN 287-1 [8.4]. The qualification is not unique for all types of welding and it is separate for butt welds, fillet welds, welding positions, layers techniques etc. The welding coordination, which is necessary for projects of execution classes 2, 3 or 4, should be done, during execution of the welding works, by welding coordination personnel having suitable experience and qualification in the welding operation. Coordinator's supervision tasks and responsibilities are described in EN 14731 [8.5]. In the above norm three levels of knowledge are defined: basic (B), specific(S) and comprehensive (C) levels. In EN 1090-2 the required level for each project is recommended, depending on the execution class, the steel quality and the thicknesses of the components to be welded. The general frame of the quality requirements is specified in EN 729 [8.6].

The tasks of the overall welding activities, which should be considered by the coordinators before welding works start, and be respected during their execution, are, as mentioned above, presented in EN 14731. Indicatively they are: (a) the confirmation of the product standards to be used, (b) the verification of the manufacturer's capability and qualification to meet the prescribed quality requirements, (c) the technical characteristics' review of the work to be executed (parent materials specification, quality and acceptance requirements for the welds, sequence of the welds, accessibility, joints preparation details), (d) the suitability of the welding subcontractor, if any, (e) the qualification of the welding personnel, (f) the suitability of the equipment to be used, (g) the production planning which includes the procedure specification, the sequence of the welds execution, the protective measures against probable environmental conditions, the preheating equipment, the arrangement of the production tests, if required, (h) the qualification of the welding procedures, (i) the welding consumables' compatibility, as well as their delivery and storage conditions, (j) the inspection before welding (joint preparation, fit-up of the components), (k) the inspection and testing during welding (essential execution parameters as welding current, arc voltage and travel speed, preheating, layers of welding, back gouging, welding sequence, handling of consumables, distortions and dimensions checking), (l) the inspection and testing after welding, (m) the corrective actions in cases of defective welds, (n) the calibration and validation of measuring and testing equipment, and (o) the preparation of the quality records for the finished construction. It is evident that, depending on the nature of the building and the quality requirements, some of the above tasks could be omitted.

8.3.2 Preparation and execution

To achieve an efficient and qualitative execution for welded connections many aspects of the procedure may be considered. The corresponding specifications, technical rules and quality requirements should be clarified, for both preparation and execution phases, before the initiation of the works. The above information, as part of the preparation phase, should be included in the welding plan, part of the overall production planning.

The content of the welding plan is described in EN 1090-2. Indicatively this plan should include: (a) the welded connections' details, with the throat thickness and the length of all welds, (b) the welding procedures' specifications, (c) the measures to be taken to avoid distortions and lamellar tearing, (d) information about preheating or post-weld heat treatment, (e) the sequence of welding, with indication of the start and stop positions, in cases where the welding cannot be executed in a continuous way, while probable turning of the components, during welding, if needed, should also be indicated, (f) the quality level requirements and the acceptance criteria, (g) information about consumables to be used, (h) the inspections' plan before, during and after welding, as well as the extent of the non-destructive testing to be executed, and (i) information about the surface treatment of the components before welding.

The recommended ends geometry for the preparation of the steel components to be welded, is indicated in numerous Tables of EN 9692-1 [8.7]. The end details are valid for all steel qualities when the usual processes of welding are applied. EN 9692-1, for each detail, indicates the detail's symbol, the geometrical data for the ends preparation, the processes to be possibly used, as well as the field of application, depending on the components' thickness.

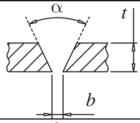
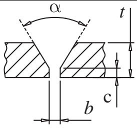
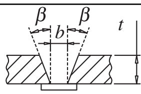
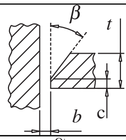
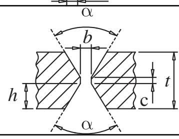
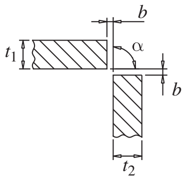
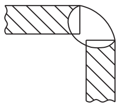
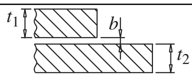
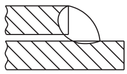
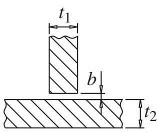
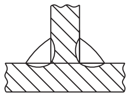
Representative geometries of the end preparation, for butt and fillet welds are shown in Table 8.3.

The surfaces to be welded should be dry, free from visible cracks and free from non-suitable materials (rust, organic materials, galvanizing), which could influence adversely the quality of the welds. Prefabrication shop primers, of a limited thickness, could be left on the fusion faces if they don't affect adversely the welding process. For projects classified in the execution classes 3 or 4, prefabrication primers should not remain on the fusion faces, unless a specific test is performed. Such a test, for assessing the influence of shop primers on the weldability, is described in EN 17652 [8.8].

The components to be welded should be brought into alignment and held in position by tack welds or external devices.

Fig. 8.3. Protection cabins for on-site welding between arch segments of a bridge

Table 8.3. Indicative types of components end preparation before welding (EN 9692-1)

	Ends preparation					
	Type of preparation	*Symbol*	*Application field*	*Figure*	*Geometry*	*Comments/ Weld illustration*
1	Single V preparation	V	$3 < t \leqslant 10$		$40° \leqslant \alpha \leqslant 60°$ $b \leqslant 4$	Applicable with backing strip
2	Single V preparation with broad root face	Y	$5 \leqslant t \leqslant 40$		$\alpha \simeq 60°$ $1 \leqslant b \leqslant 4$ $2 \leqslant c \leqslant 4$	–
3	Steep-flanked single-V preparation		$t > 16$		$5° \leqslant \beta \leqslant 20°$ $5 \leqslant b \leqslant 15$	Applicable with backing strip
4	Single bevel preparation		$3 < \mathrm{t} \leqslant 10$		$35° \leqslant \beta \leqslant 60°$ $2 \leqslant \mathrm{b} \leqslant 4$ $1 \leqslant \mathrm{c} \leqslant 2$	–
5	Double-V preparation	X	$\mathrm{t} > 10$		$\alpha \simeq 60°$ $1 \leqslant b \leqslant 3$ $c \leqslant 2$	–
6	Square preparation		$t_1, t_2 > 2$		$60° \leqslant \alpha \leqslant 120°$ $b \leqslant 2$	
7	Square preparation		$t_1, t_2 > 2$		$b \leqslant 2$	
8	Square preparation		$t_1, t_2 > 4$		$b \leqslant 2$	

(dimensions in mm)

Fig. 8.4. Preheating of a butt weld groove before welding

They should maintained, during welding, in such a way that the final dimensions of the joint are achieved and probable distortions or shrinkages remain within the specified tolerances. The components' assembly should be also arranged in such a way that the welding positions remain accessible and easily visible to the welder. The length of a tack weld should not be less than the smaller value between four times the thickness of the thicker component and 50 mm. For execution classes 2, 3 or 4 a qualified procedure should be used for the tack welding. Tack welds not being incorporated into the final welds should be removed. Otherwise they should have a suitable shape and executed by qualified welders.

The welding consumables should be stored, handled and used following the manufacturer's recommendations or according to the relevant specifications. Electrodes, especially the basic ones, must be dried before use in a temperature level between 300° and 400°C, for a time duration between two and four hours, and stored, prior to welding, in a temperature between 100° and 150°C. In case that electrodes remain unused at the end of the welding procedure, they shall be dried again, but no more than twice. Electrodes showing signs of damages or deterioration, should be rejected.

The welder and the welding place should be adequately protected against wind, rain or snow. Gas protected welding processes are very sensitive to wind actions. In important welds, specific protection measures are to be taken as shown in Fig. 8.3. In cases where the components to be welded have a temperature below 5°C, a suitable preheating is necessary (Fig. 8.4).

In cases of butt or fillet welds with significant throat thickness, the deposit of the additional welding material by the electrodes is executed in successive runs (see Fig. 8.5), each corresponding, approximately, to an indicative thickness of 5 to 7 mm.

Visible imperfections, as cracks or cavities, should be removed from each run before the deposition of the next. Precautions, during execution, are also to be taken to avoid weld spatter. If any, in execution classes 3 and 4, it should be removed.

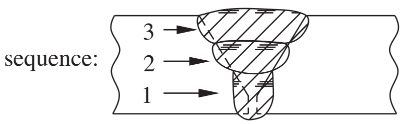


Fig. 8.5. Successive runs in the welding procedure

8.3.3 Welds imperfections

Many types of imperfections can be observed after the execution of a weld that could be attributed to human errors, non-qualified and inexperienced personnel, defective preparation of the joint area, environmental unexpected conditions, technical problems, non-appropriate electrodes etc. Imperfections are divided in defects appearing in the surface of the weld (surface imperfections) and in non-visible defects, produced inside the weld's area (internal imperfections), as well as in joint geometry imperfections, like misalignments between connected plates.

Cracks are a usual defect for both surface and internal imperfections. As cracks interrupt the continuity of the connecting material, forces can only partially be transferred, depending on the cracking length, and therefore, independently of the required quality of welding, such welds are to be rejected.

Main types of surface imperfections are shown in Fig. 8.6. They are related to insufficient or excessive throat thickness of fillet welds (Fig. 8.6a, b), excessively unequal leg lengths in fillet welds (Fig. 8.6c), excess weld metal in butt welds (Fig. 8.6d), excessive convexity in fillet welds (Fig. 8.6e), incorrect weld toe in butt and fillet welds (Fig. 8.6f, g), overlap of the welding material (Fig. 8.6h), excess penetration in butt welds (Fig. 8.6i), continuous or intermittent undercut (Fig. 8.6j), shrinkage grooves (Fig. 8.6k), incomplete root penetration for single side butt welds (Fig. 8.6l) and root concavity (Fig. 8.6m). Other types of surface imperfections, not shown in Fig. 8.6, are the stray arc and the surface pores.

EN 5817 [8.9] deals with welding imperfections and defines three quality levels, named B, C and D, providing, for each type of imperfection, the limits between levels. B is the highest quality level. In the project specification and the quality plan the required level could be agreed between contracting parts, while a higher quality level is possible to be specified for a part of the structure or for specific members. The required quality level is correlated in EN 1090-2 to the execution class in which the structure is classified. The acceptance level for each execution class is presented in 8.3.5. For usual structures, of EXC 2, quality level C is recommended.

In Table 8.4 the quality level limits provided by EN 5817, as above, are included, for the types of surface imperfections presented in Fig. 8.6 and for components thicker than 3 mm. Surface cracks or a burn-through are not permitted for all quality levels. A stray arc is permitted only in quality level D, when the properties of the parent metal are not affected. Spatter could be accepted in all quality levels, depending on the application (material, corrosion protection). Surface pores are not permitted in quality level B. For quality level C the maximum dimension of a single pore should be less than 20% of the nominal throat thickness for fillet welds (or the nominal butt welds' thickness) and not more than 2 mm. The corresponding limits for quality level D are 30% and 3 mm. EN 5817 includes also other types of imperfections. Recommendations for the detection methods which are appropriate for each type of imperfection are not included in this Norm.

Some of the limits of Table 8.4 are valid only when the corresponding imperfections could be classified as short. According to the application rule of EN 5817, in a weld of a length greater than 100 mm, an imperfection is characterized as "short" if its total length, in the 100 mm of the weld which contain the greatest number of

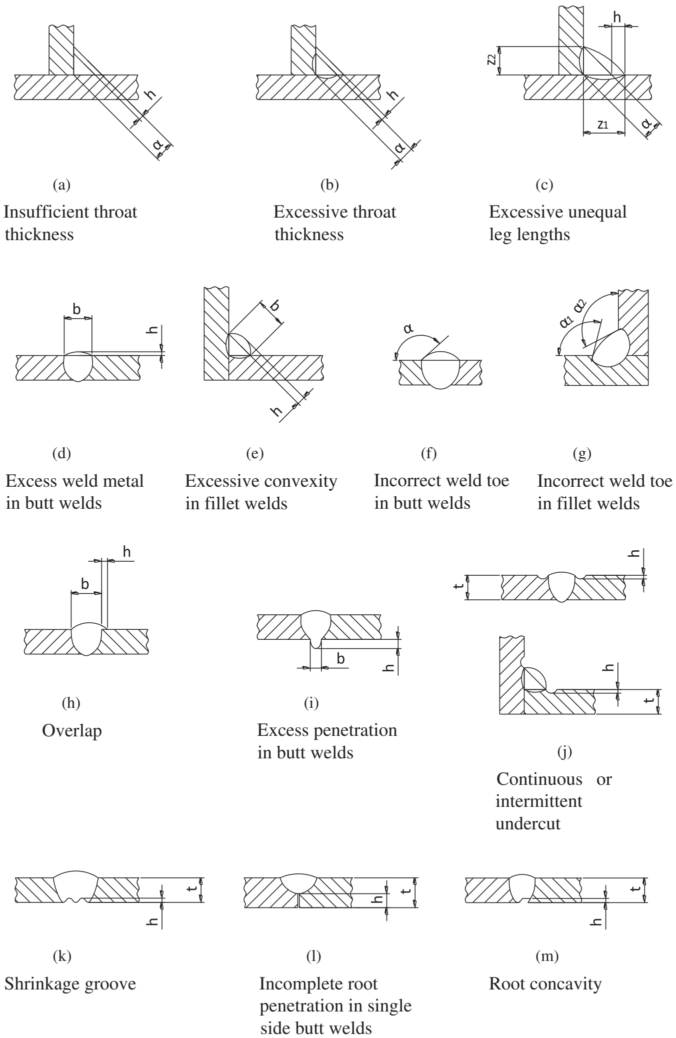


Fig. 8.6. Usual types of surface imperfections in welding

imperfections, is not more than 25mm. In welds shorter than 100 mm, "short" is an imperfection in which its total length is not greater than 25% of the weld's length. When this rule is to be applied, it is indicated with "SI" (short imperfection) in the Table. When an imperfection is repeated along the weld, it is characterized as "systematic". Systematic imperfections are only permitted in quality class D provided that the other quality requirements are fulfilled. Finally, for some types of imperfections, the indication "ST" is included in Table 8.4, meaning that a smooth transition between the weld and the parent metal, without irregularities, is required.

In internal imperfections, cracks, as already mentioned, are not permitted for all quality levels. Micro-cracks, only visible under microscope, are permitted in quality level D and could be accepted in levels C and B, depending on the type of the parent

Table 8.4. Limits of surface main imperfections for the quality levels defined in EN 5817

Type of imperfection	Designation in Fig. 8.6	Limits of imperfections for the three quality levels		
		B	C	D
Insufficient throat thickness (SI)	(a)	h ⩽ 0.3mm + 0.1α maxh = 2 mm	h ⩽ 0.3mm + 0.1α maxh = 1 mm	Not permitted
Excessive throat thickness	(b)	Permitted	h ⩽ 1mm + 0.2α maxh = 4 mm	h ⩽ 1mm + 0.15α maxh = 3 mm
Excessive unequal leg lengths in fillet welds	(c)	h ⩽ 2mm + 0.2α	h ⩽ 2mm + 0.15α	h ⩽ 1.5mm + 0.15α
Excess weld metal in butt welds (ST)	(d)	h ⩽ 1mm + 0.25b maxh = 10 mm	h ⩽ 1mm + 0.15b maxh = 7mm	h ⩽ 1mm + 0.1b maxh = 5mm
Excessive convexity in fillet welds	(e)	h ⩽ 1mm + 0.25b maxh = 5 mm	h ⩽ 1mm + 0.15b maxh = 4mm	h ⩽ 1mm + 0.1b maxh = 3mm
Incorrect weld toe in butt welds	(f)	α ⩾ 90°	α ⩾ 110°	α ⩾ 150°
Incorrect weld toe in fillet welds	(g)	α ⩾ 90°	α ⩾ 100°	α ⩾ 110°
Overlap	(h)	0.2b	Not permitted	Not permitted
Excess penetration in butt welds	(i)	h ⩽ 1mm + 1.0b maxh = 5 mm	h ⩽ 1mm + 0.6b maxh = 4mm	h ⩽ 1mm + 0.2b maxh = 3mm
Continuous or intermittent undercut (SI, ST)	(j)	h ⩽ 0.2t maxh = 1 mm	h ⩽ 0.1t maxh = 0.5 mm	h ⩽ 0.05t h ⩽ 0.5 mm
Shrinkage groove (SI, ST)	(k)	h ⩽ 0.2t maxh = 2 mm	h ⩽ 0.1t maxh = 1 mm	h ⩽ 0.05t maxh = 0.5 mm
Incomplete root penetration for single side butt welds (SI)	(l)	h ⩽ 0.2t maxh = 2 mm	Not permitted	Not permitted
Root concavity (SI, ST)	(m)	h ⩽ 0.2t maxh = 2 mm	h ⩽ 0.1t maxh = 1 mm	h ⩽ 0.05t maxh = 0.5 mm

α, nominal throat thickness of the weld
SI, short imperfection (see text)
ST, smooth transition (see text)

metal and in particular on the connection sensitivity against cracks. Porosity (gas pores) is also a usual defect in welds. It is valuated as the percentage of the pores' area against the overall examined surface (in cases of a single layer examination) and, additionally, according to the maximum dimension of a single pore. The limits for the maximum dimension of a single pore is, for quality level C, 30% of the nominal throat thickness in fillet welds (or the nominal thickness in butt welds), but not more than 4 mm, while the limit for the above single layer percentage is 1.5%. The corresponding limits for quality level D are 40%, 5mm and 2.5% while for quality level B, 20%, 3mm and 1%, respectively. Specific information is included in EN 5817 for the cases of localized or linear porosities, while, in an Annex, pictures are provided, of projected, as in radiographs, or cross-sectional areas, corresponding to different porosity percentages. Two indicative such pictures are shown in Fig. 8.7.

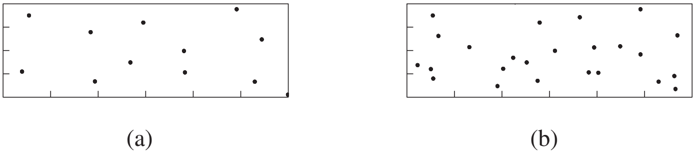


Fig. 8.7. Pores of 1mm diameter covering (a) 1.5% and (b) 3% of the total surface (EN 5817)

Other types of internal imperfections are shown in Fig. 8.8 for which the normative limits are included in Table 8.5. Such imperfections are different types of lack of fusion (Fig. 8.8a), the incorrect root gap in fillet welds (Fig. 8.8b), the lack of penetration for fillet and butt welds (Fig. 8.8c, d) and the linear misalignment between plates (Fig. 8.8e). The limits for the classification of these imperfections, shown in Fig. 8.8, in quality levels, are included in Table 8.5 and they are valid for components of thickness more than 3 mm. Finally, limits are given in the Norm for the several types of possible inclusions (solid, slag, flux, oxide, metallic other than

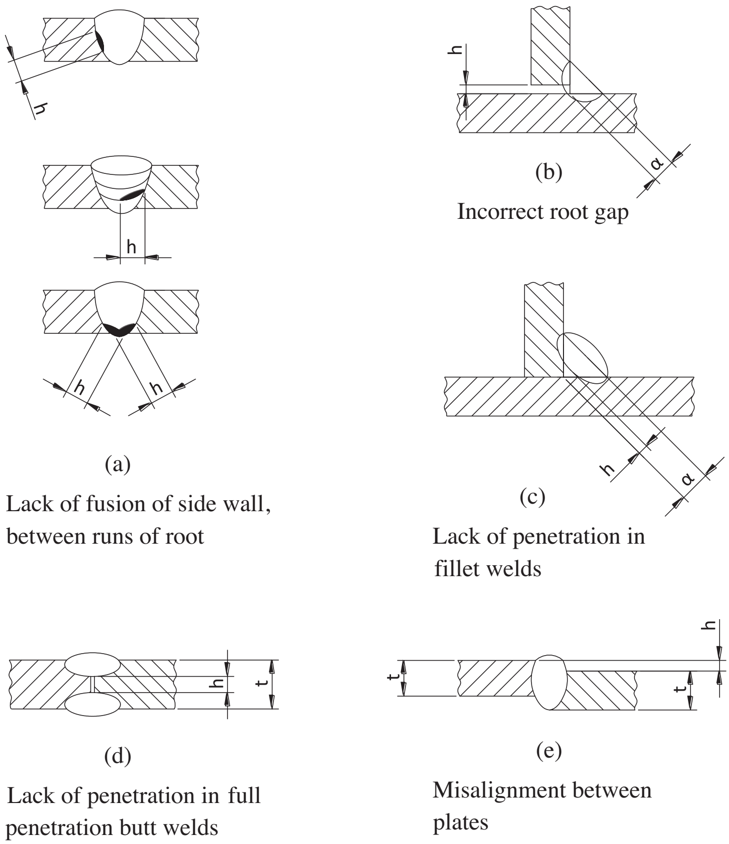


Fig. 8.8. Types of internal imperfections in welds

Table 8.5. Limits of internal imperfections for the quality levels defined in EN 5817

	Designation in Fig. 8.8	Limits for the three quality levels		
		D	C	B
Lack of fusion (SI)	(a)	$h \leqslant 0.4\alpha$ $h \leqslant 0.4s$ $h \leqslant 4mm$	Not permitted	Not permitted
Incorrect root gap	(b)	$h \leqslant 1mm + 0.3\alpha$ maxh = 4mm	$h \leqslant 0.5mm + 0.2\alpha$ maxh = 3mm	$h \leqslant 0.5mm + 0.1\alpha$ maxh = 2mm
Lack of penetration in fillet welds (SI)	(c)	$h \leqslant 0.2\alpha$ maxh = 2mm	Not permitted	Not permitted
Lack of penetration in full penetration butt weld (SI)	(d)	$h \leqslant 0.2t$ maxh = 2mm	Not permitted	Not permitted
Misalignment between plates	(e)	$h \leqslant 0.25t$ maxh = 5mm	$h \leqslant 0.15t$ maxh = 4mm	$h \leqslant 0.10t$ maxh = 3mm

α, nominal throat thickness of a fillet weld
s, nominal thickness of a butt weld
SI, short imperfection (see text)

copper) in the welds' body, depending on the inclusion dimensions in relation to the weld thickness. Copper inclusions are prohibited in all quality levels.

Multiple imperfections, where different types of imperfections occur at the same weld's cross-section, need special consideration, provided that the requirements for a single imperfection are not exceeded. Recommendations for some specific cases are included in EN 5817. In any case two adjacent imperfections separated by a distance smaller than the major dimension of the smaller imperfection, shall be considered as a single one. Finally, additional requirements are included in the above normative document for welds subjected to fatigue loads.

8.3.4 Non-destructive testing

8.3.4.1 Introduction

To detect possible welding defects, such as the ones presented in 8.3.3, different testing procedures, not requiring failure of the weld, are used, called "non-destructive tests" (NDT). The main types of such testing, besides the visual inspection, are: the magnetic particles testing, the penetrant testing, the ultrasonic method and the radiographies. For the field of application of each method reference will be made in 8.3.5.

Each method has its own capacity to detect defects of a margin of magnitude. Radiographies are able to detect imperfections between 100 and 10000 μm, the penetrant liquids between 1 and 1000 μm, while magnetic procedures between 1/10 and 1/100 μm and ultrasonic method between 1/10 and 1 μm. The field for the visual inspection is between 10 and 100 μm. In usual buildings, visual examination is extended to the whole number of welds and it is supplemented by additional testing.

The extent of this testing depends on the nature and the importance of the structure and it is determined by the regulations, the project specification, as well as by the testing and quality plans. Reference about this extent is included in 8.3.5.

The personnel involved in the non-destructive testing, visual inspection included, should be familiar with relevant specifications and rules and qualified, to an appropriate level by case, in accordance to EN 473 [8.10].

8.3.4.2 Visual inspection

The extent of the visual inspection of the welds should be defined in advance in the project specification or in the inspection plan, by the application of a standard or by agreement between the contracting parts. The inspection could be extended in the joint preparation phase, before welding starts, during welding and after welding is completed. Information about the performance of the visual inspection is included in EN 970 [8.11].

A visual inspection of the joint preparation should examine: the shape and dimensions of the joint to be executed (arrangement of the root, angle between components, uniformity of the ends preparation, alignment of the parts), the fusion faces and adjacent surfaces to be clean, the appropriate fixing of the components to be welded, following specific drawings or instructions, to avoid distortions. The examination during welding execution could inspect: that each run is cleaned before application of the next one, that there are no visible imperfections as cracks or cavities, that the transition between runs and between weld and parent metal has a satisfactory shape for melting of the next run, that the depth and shape of gouging is in accordance with the welding procedure specification.

Examination of the finished weld should inspect: (a) the weld clearness, regarding the absence of weld spatter, tool impressions or blow marks and especially the slag removal, manually or by mechanical means, (b) the profile and the dimensions of the weld. More specifically, in this field, are to be examined, in relation to the acceptance standards: the face profile and the height of any excess of weld metal in relation to the quality requirements, the regularity of the weld surface, the surface pitch in comparison to the applied pattern, the overall visual appearance as well as the consistency of the weld width over the whole length of the joint, and (c) the weld root and surfaces.

The visually accessible parts of the welds should be examined for deviations from the acceptance criteria, as for instance in the case of single-sided butt welds in which penetration, root concavity or any burn-through are possible to be inspected. In addition they should be examined, considering the acceptance criteria, undercuts and imperfections, as cracks or visually detected porosity. In the inspection is included the verification that temporary welded attachments, if prescribed, are removed and that the area of attachment is cleaned and free of cracks. In the welding of the bracing members of trusses with hollow sections, special attention should be given for circular sections to the mid-toe, mid-heel and mid-flank positions, while for square or rectangular sections to the four corner points.

For the visual inspection it is recommended that the illuminance at the weld surface should be of 500 lx and, in any case, not less than 350 lx, while the accessibility of the weld should ensure a maximum eye distance of 600 mm, under an angle of at least 30°. When specified, examination records should be included in the structure's documentation. For the inspection, appropriate examination equipment, as devices and gauges, is used.

8.3.4.3 Magnetic particles testing

Magnetic particles testing techniques are appropriate to inspect non visible surface imperfections, mainly cracks, even of a very limited width, in the order of 1/10000 mm, which are not possible to be inspected using other non-destructive methods. According to the method, very small magnetic particles are sprayed on the surface under inspection, and then a magnetic field is introduced. Cracks produce a discontinuity to this magnetic field, resulting in an arrangement of the particles along the crack.

Surfaces to be examined should be free from dirt, heavy and loose paint, oil, grease, weld spatter, scale and any material that could influence sensitivity. Cleaning and surface preparation should not be detrimental for steel or for the magnetic testing media. A typical configuration of the test is shown in Fig. 8.9. The flux current path, of length d in the figure, must be at least 50 mm plus the width of the weld and the heat affected zone, which must be included in the inspected area. The detectability of the imperfection depends on the angle of its axis, with respect to the direction of the magnetic field. To ensure detections of cracks in all orientations, the welds should be magnetized in two directions, approximately normal to each other, with a maximum deviation of 30°. The investigated width b=d/2 (see Fig. 8.9) should be overlapped by the width of the next inspected area. For usual cases the magnetic field strength should be between 2 and 6 kA/m. Documentation of the inspection is done by photographing.

EN 1290 [8.12] is related to magnetic particles testing. The specification includes characteristic arrangements of testing, for the usual cases of welds, and the field of application for each. Acceptance levels are included in EN 1291 [8.13].

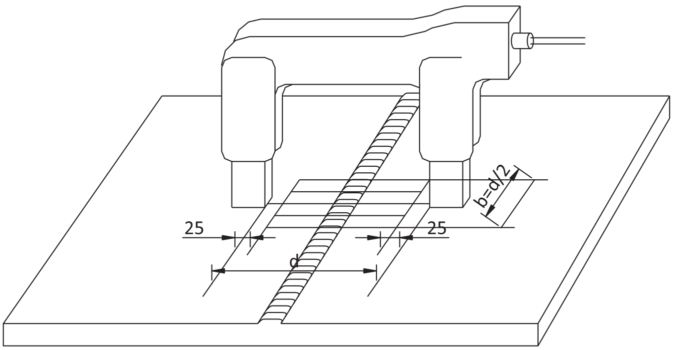


Fig. 8.9. Configuration of a magnetic welds inspection (EN 1290)

8.3.4.4 Penetrant liquids

The method of penetrants is also used to detect discontinuities as cracks, lack of fusion, porosity or laps. The penetrant, after preparation and application, enters inside discontinuities and the penetration lines appear when appropriate material spread on the tested surface. The width of the inspected surface must include at least the width of the weld plus 10 mm from each weld side.

The process sequence includes:

a) Preparation and cleaning of the surface to be detected. This cleaning must ensure that the surface is free from dirties, rust, oil, grease and any residues, in order the penetrant is free to enter in the discontinuities. For cleaning, mechanical or chemical materials or combination of both are used. Surface conditions are directly related to the minimum detectable imperfection size. Surface roughness or irregularities can cause non-relevant indications, resulting in a low probability to detect small imperfections.
b) Drying of the surface so that neither water nor solvents remain in the discontinuities.
c) Application of the penetrant. As penetrants liquid materials are used, divided in fluorescent penetrants, color contrast or dual-purpose penetrants (fluorescent color contrast). The penetrants can be applied by spraying, brushing, flooding, dipping or by immersion. The temperature of the inspected surface must be between 10^{o}C and 50°C, to minimize the moisture entering into the discontinuities. Out of these limits, specific measures should be taken. The penetration time varies between 5 and 60 min, depending on the properties of the penetrant, the temperature, the type and size of the discontinuities to inspect.
d) Excess penetrant removal, using suitable techniques and appropriate materials, as water, solvents or other hydrophilic (water-dilutable) and lipophilic (oil-based) materials. The application of the remover should be such that no penetrant is removed from the discontinuities. After removal, the surface to be inspected should be dried in a way ensuring that the penetrant, already inside the discontinuities, does not dry.
e) Application of the developer which is a material facilitating the identification of the penetration lines and discontinuities, in which the penetrant is invaded. As developers, dry powder (only when fluorescent penetrants are applied), solvent based, water-soluble or water-suspendable materials, are used. The development time varies between 10 and 30 min. The developer must be uniformly applied to the surface, and, as soon as possible, after the excess penetrant removal.
f) Inspection which is performed under appropriate viewing conditions, depending on the penetrant used. Magnification instruments and contrast spectacles aids are often used.
g) Recording of the test which could be done by a written description, sketches, adhesive tape, peel-able developer, photograph, photocopy or video.

The materials applied in a penetrant testing should be used as families of products. A family contents the penetrant material, the excess penetrant remover and the developer, which must be compatible and related between each other. Only approved product families should be used, probably produced by the same manufacturer. Each product family has as a main characteristic its sensitivity, related to the ability to detect small imperfections. Higher sensitivity materials are used to detect smaller imperfections. As penetrant inspection often requires the use of inflammable, volatile or harmful materials, appropriate precautions should be taken during the inspection procedure.

The penetrant testing is described in EN 571-1 [8.14]. EN 1289 [8.15] defines acceptance levels, depending on the length of the indication or, for non-linear indications, their major axis dimension. Information about properties for the products used in penetrant testing are provided by EN 571-3 [8.16].

8.3.4.5 Ultrasonic testing

The principle of this type of testing is to produce and propagate ultrasonic waves, dispatched through a probe, through the object to be detected, and monitoring either the transmitted signal or the signal reflected or diffracted from any discontinuity. The sound reaction is depicted on the screen of a monitor. The interpretation of the images received must be done by experienced personnel, in the frame of EN 473 [8.10], already mentioned in 8.3.4.1, able, beyond interpretation, to regulate the action range and the sensitivity setting of the equipment. The position and the magnitude of the defect are determined by measuring and evaluation of the required time for the travelling of the waves. The produced frequencies are within the range of 2 to 5 MHz, depending mainly on the thickness of the steel components (higher frequencies for thinner components).

The method is a quick procedure, primarily applied for the testing of full penetration butt welds, in ferritic steel elements, thicker than or at least equal to 8 mm. Under additional conditions it could be applied to non-ferritic steels or to partial penetration butt welds. The inspected object should be within a temperature range between 0° and 60°C. The general arrangement in the inspected area is shown in Fig. 8.10. The dimension a in the figure is the width of the inspected area (weld width, plus 10 mm from both sides). A lateral area of width b is needed to deliver sound waves from different positions (1, 2 or 3). The records from the different positions are complementary examined to determine detects extent. Imperfections perpendicular to the testing surface are difficult to be detected, therefore mainly longitudinal imperfections can be inspected. Scanning surfaces should be free from foreign materials, such as rust or weld spatter, or irregularities that may interfere with probe coupling. Waviness of the test surface should not result in a gap between the probe and the surface greater than 0.5 mm.

EN 17640 [8.17] deals with weld testing using ultrasonic waves. Four testing levels are specified in this Norm. From testing level A to level C, an increasing probability of detection is achieved by an increasing testing coverage (number of scans, surface of dressing). Testing level D may be applied for special structures. In general, testing levels are related to the quality levels of EN 5817 (see 8.3.3). Testing level A corresponds to quality levels C and D of welds to EN 5817 [8.9], while testing level B to quality level B.

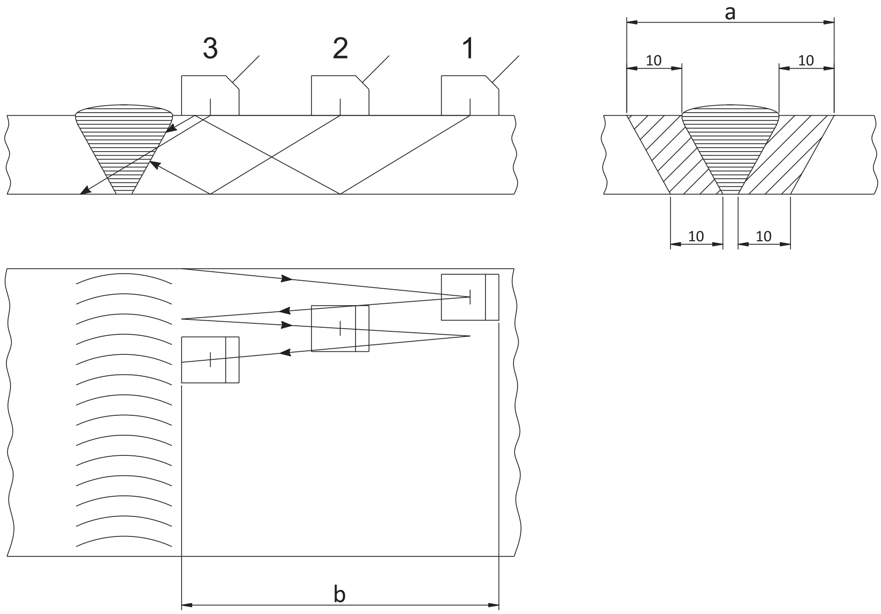


Fig. 8.10. Configuration of a weld inspection by ultrasonics (EN 17640)

8.3.4.6 Radiographic testing of welded joints

In the radiographic testing the weld under investigation is exposed to X-rays or to γ-rays, produced by a radiation source. Imperfections are impressed in the resulted radiographies. As the rays can be very injurious for the human health, strict protective measures are necessary to be taken in the area of the testing. Therefore radiographic examination cannot be performed on-site and is not usually used in building structures.

Radiographies are carried out after the final stage of manufacturing and are appropriate to detect porosity, metallic inclusions, the lack of or an incomplete fusion. Generally, surface preparation is not necessary before testing. However a clean and smooth surface is suitable to avoid difficulties in the examination of the detected defects. The maximum thickness of the material to be detected depends on the type and the size of the radiation source and could vary up to 200 mm. The films should overlap sufficiently, to ensure that the complete region is radiographed.

EN 1435 [8.18] specifies radiographic testing of welds and includes related technical information as well as sketches, concerning test arrangements for many usual types of welds. Such sketches are shown in Fig. 8.11, concerning a single side full penetration butt weld (Fig. 8.11a), and two alternative arrangements for fillet welds (Fig. 8.11b, c). The minimum source-to-object distance f is related to the object-to-film distance b (see Fig. 8.11) and the source size. For the usual cases, the source-to-object distance is of the order of 100 mm. In the sketches of arrangements, the Image Quality Indicators (IQI), verifying the image quality, should be added, which

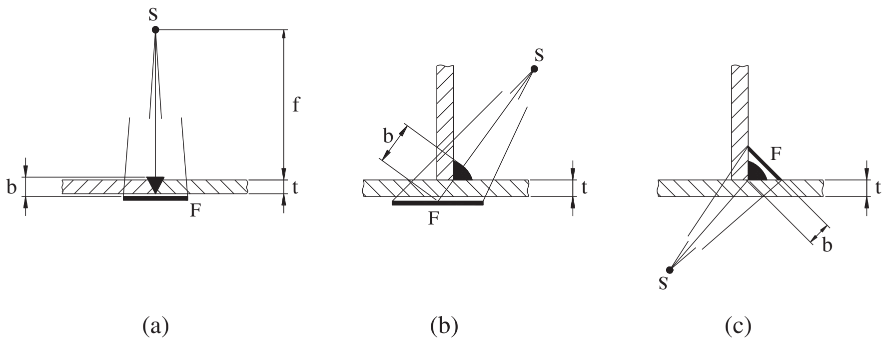


Fig. 8.11. Test arrangement for radiographic inspection (EN 1435), (a) for a single wall full penetration butt weld and (b),(c) for fillet welds (f: source-to-object distance, t: nominal thickness of the parent material and b: object to film distance)

are preferably placed on the source side of the tested element, at the center of the investigated area and in close contact with the surface of the object. EN 462 [8.19] specifies image quality indicators.

8.3.5 Inspection

All welds should be visually inspected along their entire length. In case that surface imperfections are detected, surface testing by penetrant liquids or magnetic particles should be performed on these welds. After visual inspection no further testing is required in case of execution class 1 (EXC1) structures. For the other execution classes, supplementary non-destructive testing (NDT) is required, covering both surface and internal imperfections.

The extent of the supplementary NDT depends on the execution class, the type of the welds (fillet welds, full or partial penetration butt welds), the position of the welds in relation to the welded components axis (transversal or longitudinal), the geometry of the overall welding and the utilization factor of the weld (ratio between the design value of the acting force and the design resistance of the weld). The percentage of the total number of welds to be checked is given in detail in EN 1090-2. For instance, for transverse butt welds with a utilization factor greater than 50%, a percentage of 10% for EXC2, 20% for EXC3 and 100% for EXC4 structures should be tested. For longitudinal welds and welds to stiffeners, or for transverse fillet welds in tension or shear, having a throat thickness less than 12 mm and connecting parts thinner than 20 mm, the corresponding percentages are 0%, 5% and 10%.

The joints for inspection should be selected to be representative out of the total number of welds. To this end, the type of joints, the grade of the connected steel parts, the welding equipment and the individual welders are criteria to be considered. EN 12062 [8.20] provides details about the selection of the sampling for inspection. In the execution specification (see clause 8.8) specific joints presenting difficulties to be executed, or having a high degree of utilization, could be indicated for inspection. The supplementary NDT should be performed after a minimum hold time from the welding execution, depending on the throat thickness of a fillet weld or the component thickness for a full penetration butt weld, the steel quality and the heat input ratio during execution. Detailed information is included in a Table of EN 1090-2. For usual steels (S235 to S420) this time varies from the cooling period of the weld to 40 hours.

The acceptance criteria, after inspection, are related to the levels of imperfections presented above (see 8.3.3). For structures classified in execution class (EXC) 1, quality level D is required, while quality level C for EXC2 and level B for EXC3. For structures of EXC4, quality level B+ is required, where level B+ corresponds to quality level B with additional requirements, included in a Table of EN 1090-2. When more than a simple visual inspection is needed, an inspection plan should be established which includes all different methods to be applied, as well as the scope and the sequence of the testing. After testing, a final report containing test results and the related information is to be issued. Generally, ultrasonic testing (or radiographic testing) applies to butt welds, and penetrant testing or magnetic particles inspection to fillet welds. Information about the selection of the appropriate, by case, method is given in EN 12062 [8.20], for surface or internal imperfections, depending on the type of steel (ferritic or austenitic), the type of weld (butt joint, T-joint etc.) and the thickness of the connected structural parts. To obtain the required result, a combination of different methods may, in some cases, be used. Before selecting the testing methods, the weld accessibility, the required quality level, the welding process used, the geometry of the joint, as well as the type and magnitude of the probable expected imperfections, should be considered. The personnel performing NDT and evaluating the corresponding results for the rejection or final acceptance of the welds, should be qualified, at an appropriate level, according to EN 473 [8.10], as already mentioned in the introduction.

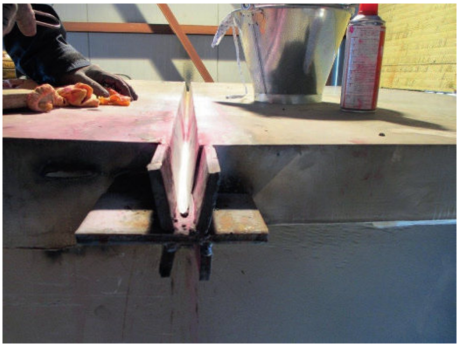


Fig. 8.12. Extension of the welding length during fabrication

When distortions at the connection area should be corrected by flame straightening, a local application of heat is introduced taking care that the maximum developed temperature in the steel, as well as the cooling phase, are, both, under control. In structures of high importance (classified in execution classes 3 or 4), a suitable procedure should be established including all procedure's details.

Defections on the welds are more often presented at the initial and the end parts of the weld. Due to this fact, in important welds, extensions could be specified, which are removed after the end of the works (Fig. 8.12).

8.4 Bolting

8.4.1 Bolt assemblies

Bolts are the common mechanical fastener used in steel structures, as well as the usual means to connect on-site structural members or subsystems of the whole structure, during the erection of the structural steelwork. For specific applications other types of bolting or mechanical fastening could also be used, as fit or injection bolts, rivets, self-tapping or self- drilling screws, fasteners for thin gauge components or for stress skin applications. The mechanical characteristics of the different bolt grades, as well as their behavior and resistance capacities are presented in chapter 5. Bolts can transfer forces perpendicular to their axis by bearing of the bolt shank to the holes' surfaces, or by friction developed between the connected parts, after preloading by tightening of the bolt. Bolts can also transfer forces applied in the direction of their axis.

Each bolt is accompanied with a nut, usually with washers, and, when necessary, with locking devices. The whole set is usually called a 'bolt assembly' and is provided by the same manufacturer. The bolt grade is marked on the bolt head and the nut. Installation of the bolts should be such that, after erection, the grade designations remain visible in case of inspection. Nuts shall run freely on their bolt and this is to be checked before installation.

Washers, as already mentioned in chapter 5, are not, in general, required in non-preloaded bolts, in normal round holes. In preloaded bolts, of a 10.9 quality, two washers, under the bolt head and the nut, are required. For 8.8 preloaded bolts one washer is required, under the bolt head or the nut, whichever is to be rotated during installation. However, in all cases, the designer could indicate the use of additional washers to reduce local damages to the structure, as, for instance, in the case of thick coatings. Washers are also used in bolts placed in slotted or oversized holes. To adjust the grip length, it is accepted to use additional washers, up to a maximum total number of three, and a maximum total washers' thickness of 12 mm. The washers are placed on the not turned side. Washers should not be thinner than 4 mm.

In specific cases, where the connection could be subjected to impact loads or significant vibrations, the assembly could be equipped with locking devices, such as prevailing torque nuts, or types of bolts preventing loosening of the assembly. In general, the project specification should specify if, additional to tightening, other measures are to be taken to secure nuts. In no case the head of bolts or the nuts should be welded to the connected parts, in order to be secured, unless otherwise specified. Preloaded bolts do not need additional locking devices.

The bolt length and the unthreaded part of its shank should, after application, fulfill the following rules : (a) the length of the protrusion should be at least equal to one thread pitch, measured from the outer surface of the nut to the end of the bolt, (b) for non-preloaded bolts one full thread should at least remain clear, between the bearing surface of the nut and the unthreaded part of the shank, (c) for preloaded bolts the above length should be at least four full threads, and (d) in connections calculated using the shear capacity of the unthreaded part of the shank, allowance should be available for the tolerances on the length of the unthreaded part of this shank. In general the minimum nominal bolt diameter used for structural bolting is 12 mm.

For non-preloaded bolts, each bolt should be sufficiently tightened. As sufficient tightening is considered the one applied by the effort of one man, using a normal sized spanner, without an extension arm. To achieve a uniform tightening of all bolts in a bolted connection, this tightening should be executed in more than one cycle, starting from the most rigid bolts of the connection. In a cover plate of an I-section joint, the most rigid bolts are in the middle of the connection, while in the case of end-plates the most rigid bolts are beside the flanges. In a bolted connection of significant dimensions in both main directions, tightening starts from the center of the connection and ends at the bolts of the perimeter. Care has to be provided to avoid overtightening of the bolts, especially in the cases of short and M12 bolts.

Separate steel components, belonging to the same connection, should not differ in thickness by more than D=2mm (see Fig. 8.13) for non-preloaded, and by more than 1mm for preloaded bolts. For differences larger than the above limits, packing plates should be used, the thickness of which should not be less than 2 mm. The thickness of the plates should be selected so that no more than three packing plates are needed. In case of aggressive environment, closer contact should be required in the project specification, to avoid cavity corrosion. In non-aggressive conditions and for constituent parts having a thickness greater than 4mm for plates and greater than 8 mm for members of cross-sections, a residual gap of up to 4 mm could be accepted at the edges of the connection, provided that contact bearing is achieved in its central part. Packing plates should have compatible corrosion behavior and mechanical strength with the main elements of the connection.

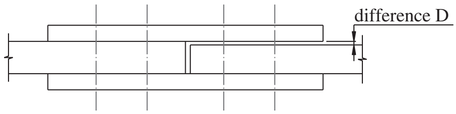


Fig. 8.13. Difference in the thickness between connected parts to be probably covered by packing plates (EN 1090-2)

Bolts could be installed without a surface protection (“black”), in cases where they are not exposed to external environmental influences, or be galvanized with a protection thickness of at least 40μm. Galvanizing should be executed by the bolts’ manufacturers. Hot dip galvanized bolt coatings should conform to EN ISO 10684 [8.21].

Foundation bolts should have mechanical properties in accordance with EN ISO 898-1 [8.22], which concern usual bolts, as above. Hot rolled steel or reinforcing steel bars could also be used.

8.4.2 Tightening of preloaded bolts

The contact surfaces of the connected main parts should be cleaned from any undesirable material, which could reduce the friction between them, such as oil, dirt, rust or paints. During cleaning, care is to be taken to avoid damage or smoothing of the roughened surfaces. The slip factor is generally determined by test. It is accepted that the usual surface treatments, mentioned in Table 5.8 of chapter 5, provide at least the slip factor values indicated in this Table, without testing. In EN 1090-2 a test and a related procedure are described, to determine the slip factor under the existing surface conditions.

Alternative tightening methods possible to be used during construction, have already been presented in chapter 5. In the usual cases, in order to achieve the preloading force, a torque moment is applied to the bolt, through a torque wrench. Torque wrenches should be reliably calibrated. The target of the calibration is to record the torque values applied by the wrench, needed to achieve the specified preload tension in the bolt. Wrenches used in the torque method shall be able of an accuracy of 4% (according to EN ISO 6789 [8.23]). The wrench should be checked for accuracy at least weekly, while in the case of pneumatic wrenches every time the hose length is changed. For wrenches used for the first step in the combined method, the requirement for the accuracy is 10% and the calibration period at least once per year, unless the wrench manufacturer specifies a shorter time.

Tightening is to be applied by rotation of the nut. In cases where the access to the nut side is difficult, it is permitted that tightening is carefully performed by rotation of the bolt's head. The as-delivered calibration conditions for the wrenches are valid for tightening by rotation of the nut. If the tightening is done by rotation of the head, a new calibration is required. As already mentioned for the non-preloaded bolts, tightening should be carried out in more than one cycle of preloading, starting from the most rigid part of the joint to the less rigid, as well as from the center to the perimeter of the connection.

Checking of preloading should be done in case of an unforeseen event (significant impact, earthquake, overloading), during application. In case of a significant delay in the procedure, under unfavorable environmental conditions which could alter the lubrication performance, possible alterations should be checked. The potential loss in the specified preloading, due to relaxation or creep of the surface coatings, is already considered in the tightening methods. Attention should be paid in cases of thick surface coatings.

EN 1090-2 includes, in an Annex, a test method to determine the torque values which, under site conditions, ensure that the required preloading is reliably achieved. During the test the bolt tension force in relation to the applied torque is measured, as well as the corresponding relative rotation between nut and bolt, through a mechanical device in accordance with EN 14399-2 [8.24]. An analytical procedure for the evaluation of the test results is also included in this Annex. For the testing, representative assemblies are used. Attention is to be given to the conditions of the fasteners used, and in particular to the performance of the lubrication, especially when they are left exposed to unfavorable external conditions on-site, or when left stored for a significant time.

In case of the direct tension indicator (DCI) method, compressible washer-type indicators are used, having protrusions on their surface. When preloading is applied, the protrusions yield and the washer become plane, when the specified preloading is achieved. Washers-indicators are placed under the bolt head (Fig. 8.14a), when the tightening is, as usual, applied by rotation of the nut. When the access to the bolt heads, for inspecting indicators gap, is limited, the washer could be placed under

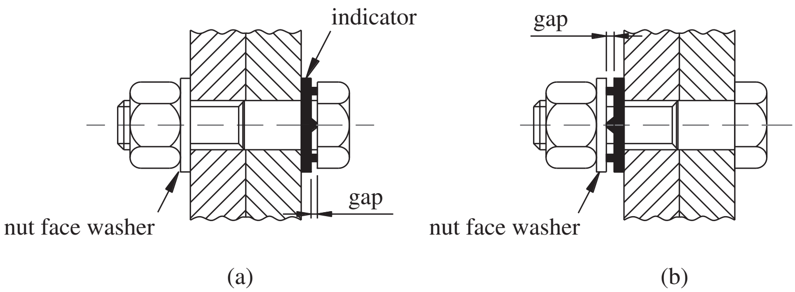


Fig. 8.14. Direct tension indicator (DC) (a) under the bolt head and (b) under the nut with additional plane washer

the nut. In this case a usual plane washer is additionally used, between indicator protrusions and the nut (Fig. 8.14b).

8.4.3 Specific fasteners

8.4.3.1 General

Bolts (preloaded or not) cover in practice all usual applications of steel structures concerning mechanical on-site fastening. However, in specific cases, or in prescribed parts of structures having specific constructional requirements, alternative types of fasteners could be used, the most usual of which are shortly presented below. In all cases, specific fasteners should be used in accordance with the product manufacturer recommendations.

8.4.3.2 Fit bolts

In fit bolts, which has already been mentioned in chapter 5, the nominal holes' diameter is equal to the shank diameter of the bolts. No relative movement between connected parts is possible, therefore they are applied when this limitation is considered as essential for the behavior of the structure, and is recommended in the project specification.

The nominal diameter of the shank is 1 mm greater compared to the threated part. The length of the threated part of the shank, included in the bearing length of the bolt, is also limited and should not exceed one third of the thickness of the connected plate, nearest to the nut (Fig. 8.15a). Fit bolts should be placed without the necessity of excessive force in order to limit damages in the thread.

8.4.3.3 Hexagon injection bolts

Injection bolt is a different type of a slip resistant bolt, in both serviceability and ultimate limit situations, without the necessity of preloading. However they could be applied as preloaded or not. In this type of bolts the clearance between the bolt and the inner surface of the hole is filled by a resin, injected through a small hole existing

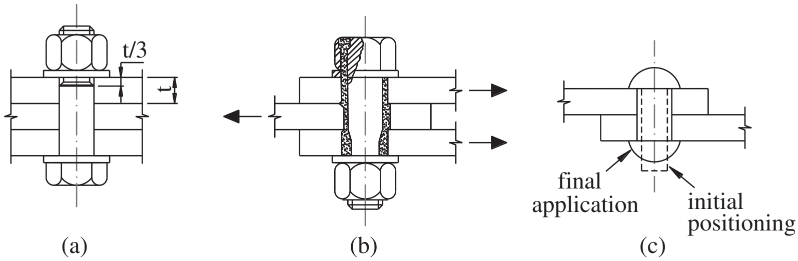


Fig. 8.15. Specific mechanical fasteners: (a) fit bolt, (b) injection bolt, (c) hot rivet

in the head of the bolt (Fig. 8.15b). The clearances of the holes are taken equal to 2 mm, for the smaller bolt diameters, while for bolts of 27 mm diameter or more, equal to 3 mm.

8.4.3.4 Hot rivets

As already mentioned in chapter 5, hot riveting, as a fastening method, has been actually substituted by bolting. However, EN 1993-1-8 [8.25] gives information about the strength and the other characteristics of the rivets to be used in the verification of existing structures, as well as in the limited cases of new applications.

The cold rivet has a diameter smaller (usually 1 or 2 mm) than the corresponding hole, and one head. The rivet length should be sufficient in order to, when installed, under pressure, in the warm situation, the hole becomes totally filled and a second head, having the prescribed dimensions and geometry, is formed (Fig. 8.15c). When, before riveting, the connected elements are placed together, in a firm contact, the maximum acceptable eccentricity, between holes corresponding to the same rivet, is 1 mm. To be adapted to this requirement, reaming is permitted. However, in this case, it has to be examined if a rivet of a greater diameter should be used.

Riveting is usually carried out using machines of a steady pressure type. After application, the machine pressure should be maintained in place for a short time, until the red head becomes black. A rivet not applied immediately after its heating, is not permitted to be re-heated.

The rivet head should be centered in relation to the shank axis. The maximum accepted eccentricity is 15% of the final diameter of the rivet (hole's diameter). Eccentric rivets and rivets presenting heads with cracks or pits or a failing contact with the connected parts should be removed and replaced. The full filling of the holes is verified with the aid of a short hammer, lightly tapping the rivet heads and checking, by experienced persons, movements, vibrations and the sound produced.

8.4.3.5 Shear connectors

References to shear connectors are included in 7.2.3.

8.4.3.6 Fasteners of thin gauge components

As gauge components are considered steel plates or members with cross-sections having a thickness up to 4 mm. The most common type of such fasteners are self-tapping and self-drilling screws. They are mainly used in small and secondary structures, designed exclusively by thin gauge elements as above, as well as for the connection of the roofing sheeting profiles to the structural members. They are usually associated with sealing washers and are located in the valley of the corrugations, unless otherwise is specified.

8.4.4 Inspection

Bolted connections should be inspected before the final approval of the steelwork, at least for all structures classified in execution classes 2, 3 or 4. For all kind of bolts a visual check of the local alignment and the packing of the connected parts is to be made. Attention should be paid for missing bolts or defective alignment in connections including an important number of bolts, in which, in order to accelerate the initial phase of erection, only some of the bolts are provisionally placed. For the inspection, a number of bolts is selected, on a random basis, representative of the overall bolted connections, using as criteria the connection type, the different fasteners lots, bolts classes, types and sizes and equipment used.

For the preloaded bolts the minimum number of bolts to be inspected is taken equal to: (a) the 5% of the total number of bolts, in structures classified in execution class 2 (EXC2, see 8.1) and for checking the second tightening step, in bolts tightened following the torque or the combined method (see chapter 5), or tightened following the direct tension indicator (DCI) method, (b) in structures classified in EXC3 or EXC4, 5%, of the total number of bolts, for verifying the preloading of the first step, and 10% for the second step when the combined method is used, and (c) in structures of the execution classes 3 or 4, 10% for the second step of the torque method and for the DTI method.

The inspection could be carried out, unless otherwise specified, following the procedure included, in an Annex of EN 1090-2. The procedure includes a graphic sequential method, according to the principles of ISO 2859-5 [8.26], having as purpose to give rules for a progressive elaboration of the inspection results. The form of the graph is shown in Figure 8.16 where in the horizontal axis the inspected number of bolts is plotted and in the vertical axis the number of the defective bolts. The graph defines three zones: the acceptance zone A, the rejection zone C and the indecision zone B (Fig. 8.16). Two different sequential plans are proposed: the plan type A for use in structures of execution classes 2 and 3 and the more demanding plan of type B for structures of the execution class 4. The graph type A is to be used for a number of bolts between 5 and 16 while for the graph type B between 14 and 40.

In case that the inspection results lead to the acceptance area no further samples are required. If the inspection lot leads to the rejection area, all bolts of the structure should be removed and reinstalled. In case that the negative result derives from a type A graph, it is permitted to enlarge the inspection using, before rejecting, a type B graph with additional bolts. For an initial lot giving a result in the indecision area

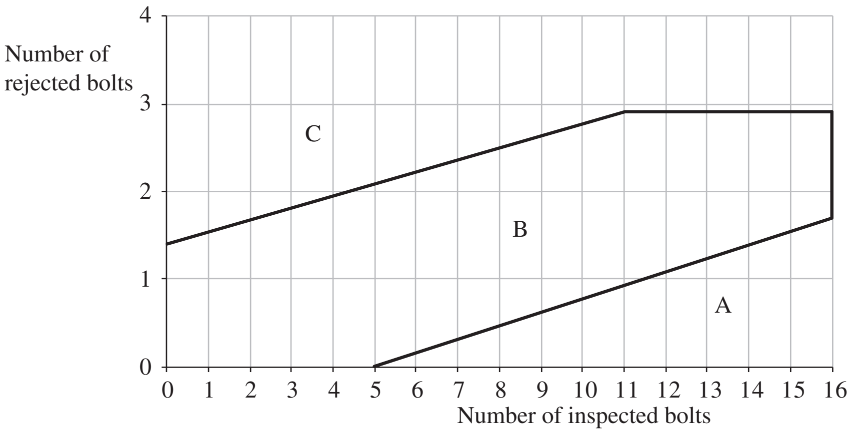


Fig. 8.16. Diagram type A for bolts acceptance after inspection (A acceptance zone, B indecision zone, C rejection zone)

a next lot of bolts should be inspected and a cumulative plot, including bolts of both lots, should be established. The procedure is continued until the cumulative plot ends in an acceptance or rejection result. In EN 1090-2 criteria for the rejection of a bolt assembly are given for all tightening methods.

Information about the acceptance inspection of grade 10.9 structural fasteners intended for controlled tightening is provided in reference [8.27].

8.5 Corrosion protection

8.5.1 Introduction

Unprotected steel exposed to the atmosphere, immerged into the water or the soil, reacts with surrounding materials and is, therefore, subjected to corrosion. This exposure leads to a progressive weakness of the cross-section, during the life of the structure, and probably to damage. Atmospheric corrosion is a process taking place in a film of moisture in the metal surface, which may be so thin that it is invisible with a naked eye. To avoid corrosion, protective measures should be taken covering the whole lifetime of the structure. Apart from protective methods applied in specific cases (chemical, mechanical, electrical, through microorganisms) the anticorrosion protection, in the usual structures, is ensured mainly by paint systems or, in some cases, by hot dip galvanizing and similar protection methods.

In general, paint systems are effective for a shorter time compared to the structure's lifetime, therefore consideration should be taken, during the design of the structure, for the possibility of their maintenance and renewal. In cases where structural members are not accessible after completion of the works, specific care should be given for a sufficiently long protection time. When this cannot be achieved by coatings, alternative measures should be taken, as for instance manufacturing these members from corrosion-resistant steel.

EN ISO 12944 [8.28] deals with the protective paint systems against corrosion and specifically with the types of environmental corrosivity, the preparation of the surfaces to be paint, the alternative protective paint systems that could be used, as well as their durability and application procedures. It covers both new and maintenance paint works.

The required durability of the protective paints, in relation to the ambient corrosivity, quantitative and qualitative, influences the selection of the appropriate type of paint and its thickness. Three durability levels, expressing the coating active life before the first major painting maintenance, are specified in the above Norm, expressed in terms of time ranges: low (L) durability, for an active life from 2 to 5 years, medium (M) durability, for a life between 5 and 15 years and high (H) durability for an active life of more than 15 years. Durability in the Norm is a technical normative time period, expresses an indication about the expected life time of the paint and, in any case, is not a guarantee time. The parameters influencing durability are the type of the paint system, the design and mainly the detailing of the structure, the conditions of the steel surfaces before preparation, the quality of the surface preparation, the state of joints, edges and welds before preparation, the conditions during the application of the paint, as well as the exposure conditions after application. The aforementioned parameters will be commented in the next clauses.

8.5.2 Types of environments

The environmental conditions and more specifically the environmental factors which promote corrosion (environmental corrosivity) represent the main parameter for the selection of the appropriate protective paint system. The corrosion rate is increased by: (a) an increased relative humidity. It is known, from practice, that significant corrosion is produced in ambient with relative humidity above 80% and temperature above 0°C, (b) the occurrence of condensation, when the steel surface temperature is below the temperature at which moisture in the air condenses out on to a solid surface (dew point of temperature), and (c) an increase in the amount of the atmospheric pollution, where the corrosive pollutants can react with steel and may form deposits on the surface.

The atmospheric humidity and the air temperature in a particular region depend on the prevailing weather conditions (climate) of this location. A climate parameter critical for the ambient corrosivity is the time of wetness, defined as the period per year during which a metal surface is covered by a film of electrolyte, able to cause atmospheric corrosion. This time can be approximately calculated by summing up the hours during which the relative humidity is more than 80% and the temperature above 0°C. EN 12944-2 [8.29], in an indicative way, gives, in an Annex, an estimation of the above period of time for six different climate conditions. In an extremely cold climate, with mean annual temperature values, -65°C as minimum and +32°C as maximum, and a highest temperature of 20°C coexistent with a relative humidity of more than 95%, the time of wetness is less than 100 hours yearly. In a warm and damp climate, where the above temperatures are +5°C, +40°C and +31°C respectively, the wetness time is between 4200 and 6000 hours.

For normative purposes, EN 12944-2 classifies environments into five atmospheric-corrosivity categories: (a) very low corrosivity (C1), for locations without pollution, (b) low corrosivity, C2, corresponding to atmospheres with low level of pollution, mostly in rural areas, (c) medium corrosivity, C3, corresponding to urban and industrial atmospheres, with moderate sulfur dioxide pollution, as well as for coastal areas with low salinity, (d) high corrosivity, C4, for industrial areas and coastal areas with moderate salinity, and (e) very high corrosivity category, C5, divided in two subcategories C5-I (industrial), corresponding to industrial areas with high humidity and aggressive atmosphere, and C5-M (marine), corresponding to coastal and offshore areas with high salinity. Corrosivity is expressed in terms of thickness loss of standard specimens, after the first year of exposure. As an example, the thickness loss of non-protected steel in a C2 environment varies between 1.3 and 25 μm per year, while for a C5 category between 50 and 80 μm yearly.

The recommended way to determine thickness loss is the exposure, in the ambient conditions, of a standard specimen. In the absence of more accurate information, the corrosivity category may be estimated on the basis of the typical environments described above and included in Table of EN 12944-2. It is to be noted that, extrapolation from the normative values in order to estimate the thickness loss for a longer period (more than one year) is not reliable.

The location of a structural element in the building influences corrosion. Members exposed to open air, rain, sunshine and pollutants, in the form of gases, affect corrosion. Under cover, climatic influences are reduced. Specific conditions and rules exist for steel structural members immerged in water (as river installations, hydroelectric power plants, sea water offshore platforms) or buried in soil (as steel pipes, piles, tanks), where corrosion is dependent on the soil mineral, water and oxygen content. Additional information about the above specific ambient conditions is not included in this book.

Corrosion of steel members located inside buildings and protected from the outside conditions is, in general, insignificant. However local conditions around a constituent element of a structure, or climate conditions inside the building, as for instance in the case of indoor swimming pools with chlorinated water, can influence decisively corrosivity. Cooler areas of structures could be subjected to unfavorable corrosive conditions as the result of seasonal formation of condensation. Members with hollow sections that are totally sealed, are not subjected to any internal corrosion. In this case discontinuous welds at their ends should be excluded.

Besides the general environmental conditions, local effects could influence further the demands on protective paint system performances. Such actions could be of a chemical character, produced by pollutants (acids, alkalis, organic solvents, aggressive gases) and related to the operation of a plant (dye mills, oil refineries, wood-pulp works), or of a mechanical character, due to particles, as for example a sand action entrained by wind in regions near deserts, or could be influences related to medium (between 60°C and 150°C) or high (between 150°C and 400°C) temperatures, as it is the case in steel chimneys or flue gas ducts.

8.5.3 Surface preparation

The main objective of the surface preparation is to remove all injurious matter, and to obtain a surface favorable for a satisfactory adhesion of the priming paint to the steel. Surface preparation also assists to reduce the amount of contaminants that initiate corrosion. Materials to be removed are the mill scale, the heavy oxide layer, formed during hot fabrication or heat treatment of steel, rust, oils, grease, dirt, existing paints, as well as any type of contaminants. The personnel performing surface preparation works should have sufficient technical knowledge and suitable equipment. The surfaces to be cleaned should be easily accessible and sufficiently illuminated. EN 12944-4 [8.30], and ISO 8501 [8.31] are related to the steel surface preparation.

The procedure to be applied for the surface preparation of uncoated steel depends on their state. Four classes of statement are specified in ISO 8501-1 [8.31], designated as rust grades, A, B, C and D, that are determined by a written description together with representative photographs. The description of the above rust grades is included in Table 8.6.

Table 8.6. Rust grades of unprepared surfaces according to EN 8501-1

Rust grade	*Description*
A	Steel surface largely covered with adhering mill scale but little, if any, rust.
B	Steel surface which has begun to rust and from which the mill scale has begun to flake.
C	Steel surface on which the mill scale has rusted away or from which it can be scraped, but with slight pitting visible under normal vision.
D	Steel surface on which the mill scale has rusted away and on which general pitting is visible under normal vision.

Surface imperfections and irregularities, at the positions of the welds or at the ends of the members, should be repaired, depending on the required preparation grade. Such imperfections are welding spatter, weld surface irregularities, welds porosity or undercuts, irregular edges made by punching, shearing, sawing or drilling, thermally cuts, surface pits or craters, grooves etc. Three preparation grades are specified in EN 8501-3 [8.32], P1 to P3, with increasing strictness from P1 to P3, depending on the expected lifetime of the paint and the environmental corrosivity. Grade P1, light preparation, corresponds to cases where no preparation or only a minimum preparation, before application of the paint, is needed. Grade P2, thorough preparation, where most imperfections are remedied and P3, very thorough preparation, where surfaces are free of significant visible imperfections.

Tables of EN 8501-3 include guidelines for the necessity of the imperfections repair, depending on the type of imperfection and the preparation grade, as above. In EN 1090-2 preparation grade P1 is recommended in cases of: (a) low active paint lifetime (2 to 5 years), see 8.5.1, and ambient corrosivity C1 to C4, (b) medium paint lifetime (5 to 15 years) and corrosivity category C1 to C3, and (c) high lifetime (more than 15 years) and corrosivity C1 or C2. In all other cases preparation grade P2 is to be applied. Preparation grade P3 may be specified only for special structures. Specific rules exist for other than bare uncoated steel types of surfaces, as for example hot-dip-galvanized surfaces, coated with zinc or zinc alloys, see 8.5.7, or surfaces painted with a prefabrication primer, see 8.5.4, after cleaning.

When selecting a surface preparation method, it is necessary to consider the preparation grade specified, in order to obtain a suitable surface cleanliness and, if additionally required, a surface roughness appropriate for the coating system to be applied. Three types of cleaning methods could mainly be distinguished: (a) water cleaning with probable simultaneous use of solvents or chemical action, (b) mechanical cleaning that could be performed by using hand-tools, power-tools or by blasting, and (c) flame cleaning.

Water cleaning consists in directing a jet of clean, fresh water on the surface to be cleaned. The water pressure to be applied, as well as the necessity to add detergents, depends on the nature of the contaminants to be removed. Solvents, emulsions or acid and alkaline materials could be applied to the surface by case, followed by a rinsing with clean and fresh water (hot or cold). In flame cleaning, a flame jet passes over the surface to be prepared. Mill scale and rust are removed by the effect of the jet and the action of the heat. Prior to flame cleaning heavy layers of rust should be removed by chipping. After that, surfaces should be cleaned by mechanical means to remove any remaining dust and contaminants. A surface cleaned in this way, viewed without magnification, shall be free from mill scale, rust, paint coatings and foreign matter. Any remaining residues should be seen only as shades of different colors on the surface. Related indicative photos are included in EN 8501-1 [8.31]. Flame cleaning is designated by FI.

Mechanical cleaning using hand-tools, such as brushes, spatulas, scrapers and hummers or power-tools, such as rotating brushes, grinders and guns, was widely used in the past. However, other methods are not considered effective as blasting, and could be applied when blasting cannot or is not indicated to be used. Prior to this type of cleaning, any heavy layer of rust should be removed by chipping. Visible oil, grease and dirt should also be removed. After a hand or power tool cleaning, the surface should be cleaned from loose dust and debris. Mechanical cleaning using tools, by hand or power operated, is designated by St. Two levels of cleaning are specified: thorough (St2) and very thorough (St3). In St2 the surface, viewed after cleaning without magnification, should be free from visible oil, grease and dirt, as well as from poorly adhering mill scale, rust, paint coatings and foreign matter. In St3, the same criteria apply as for St2, but the surface should be treated much more thoroughly to obtain a metallic sheen. Grade St1 corresponds to a surface non suitable for any type of paint. In EN 8501-1 photographs are also included corresponding to the above levels of cleaning.

In blast cleaning an air stream, including a solid, abrasive material, is directed, at high velocity, from the nozzle to the surface to be cleaned. Blast cleaning could be dry or wet, with the addition to the stream of a liquid and operated in fixed installations or by mobile units. The abrasive materials can be metallic or non-metallic, spherical or polyhedral, used many times or once. As in the previous methods, prior to the blast cleaning heavy layers of rust should be removed by chipping as well as visible oil, grease and dirt, while after blasting the surface should be cleaned from loose and debris. This type of cleaning is designated as "Sa". Four levels of cleanliness are specified: light blast cleaning (Sa 1), thorough cleaning (Sa2), very thorough blast cleaning (Sa2 $\frac{1}{2}$) and blast cleaning to visually clean steel (Sa3).

The characteristics of the above different cleaning levels are specified in EN 8501-1, supplemented by representative photographs. The description of these levels is included in Table 8.7. In EN 12944-4 [8.30] the features of the prepared surfaces, according to the different cleaning methods, are tabulated for the cases of a primary (overall) preparation, leading to bare steel, and of a secondary (partial) preparation leaving on the surfaces sound parts of organic and metal coatings. In the projects of steel buildings the Sa2 $\frac{1}{2}$ preparation level of the structural elements is usually applied.

Table 8.7. Levels of blast cleaning (EN 8501-1)

Level		*Description*
Sa 1	Light blast cleaning	When viewed without magnification, the surface shall be free from visible oil, grease and dirt, and from poorly adhering mill scale, rust, paint coatings and foreign matter.
Sa 2	Thorough blast-cleaning	When viewed without magnification, the surface shall be free from visible oil, grease and dirt, and from most of the mill scale, rust paint coatings and foreign matter. Any residual contamination shall be firmly adhering.
Sa $2^{1/2}$	Very thorough blast-cleaning	When viewed without magnification, the surface shall be free from visible oil, grease and dirt, and from mill scale, rust, paint coatings and foreign matter. Any remaining traces of contamination shall show only as slight stains in the form of spots or stripes.
Sa 3	Blast-cleaning to visually clean steel	When viewed without magnification, the surface shall be free from visible oil, grease and dirt, and shall be free from mill scale, rust, paint coatings and foreign matter. It shall have a uniform metallic colour.

8.5.4 Paint systems

8.5.4.1 Paints

Paints consist essentially of a continuous material, the binder, film forming in liquid phase, in which a pigment or a combination of pigments is dispersed. The binders are non-evaporated materials consisting the basis of the paint. They must have a high adhesion performance and the capacity to resist efficiently against environmental conditions, as well as against thermal, chemical and mechanical actions. The pigments are organic or inorganic matters dispersed in the binder, in order to offer certain properties, such as color and corrosion inhibition, as well as opacity, viscosity and durability. The solvents are added to the mixture to dissolve the binder and to facilitate the application of the paint, as they increase fluidity.

Paints could be classified according to different criteria, such as the chemical composition of the binder or of the pigments, the capacity to resist against types of environment, the coating processes, the way of drying etc. Another classification could be based on the reversibility or not of the solvent evaporation. In reversible coatings the film dries by solvent evaporation but the process is reversible, the film can be re-dissolved in the original solvent at any time. This is the case of paints used in indoors swimming pools where the water is heated and chlorinated. As binder, chlorinated rubber is used. The paint has the capacity to resist the cyclically produced water and chloride vapors. However the majority of the paints are irreversible. The film dries by solvent evaporation, followed by a chemical reaction, but the film cannot be dissolved again in the original solvent.

In some paints instead of a solvent, water is used in which the binder is dispersed. The coating film hardens by evaporation of the water and coalescence of the dispersed binder to form a film. On this basis paints could be divided in solvent-borne, water-borne or solvent-free paints. Types of paints are the so-called 2-pack paints, produced by mixing of two different materials before use. Their first component could be polymers reacting with suitable curing agents.

The drying time of a coat varies between one and seven hours (indicative values), depending on the film thickness, the nature of the paint, the air movement in the member's position and the temperature. Drying can take place even below 0°C, although at low temperatures the drying speed is much lower.

As binder, materials of several types are used, like acrylic polymer, vinyl polymer (PVC), alkyd, urethane alkyd, epoxy ester, polyurethane resin, chlorinated rubber. These materials are the most typical binders. However, new paints can be produced and distributed under the condition that they will be verified following procedures briefly commented in 8.5.6. The main properties of the different generic types of paints are presented in Table 8.8, taken from EN 12944-5 [8.33] which is the part of EN 12944 dealing with paints and paint systems.

8.5.4.2 The systems of paints

Paints are applied by successive coats (films), where the thickness of each, measured in dry condition (dry film thickness), varies usually between 40 μm and 80 μm. The initially applied to the bare steel painting is the "priming coating", while "primers" are the related paints, formulated for use as priming coats on prepared surfaces. Priming coating has, in the usual cases, a thickness of 60 to 100 μm, and is applied on one or two coats. Priming coating is covered by the subsequent coats which are divided in intermediate and final or top coats. Intermediate coats provide to the surfaces a permanent protection, facilitate adhesion with subsequent coats and increase the total thickness of the paint. Final coats provide protection to the intermediate coats, as well as the final color of the structure. Priming and subsequent coats are in general of different chemical compositions, but compatible between each other, in the sense that they can be used together in contact, without undesirable effects. The entity of successive and mutually compatible coats, having their specific composition characteristics and possessing the individual for each coat thickness, as well as their total thickness, constitute a "paint system".

Two main categories of primers could be distinguished, depending on the type of the pigment they contain: (a) zinc-rich primers, designated as Zn(R), are those in

Table 8.8. General properties of different generic types of paints (EN 12944/5)

Suitability ■ Good ▲ Limited ● Poor — Not relevant	*Poly (vinyl chloride)*	*Chlorinated rubber*	*Acrylic*	*Alkyd*	*Polyurethane, aromatic*	*Polyurethane, aliphatic*	*Ethyl zinc silicate*	*Epoxy*	*Epoxy combination*
	(PVC)	(CR)	(AY)	(AK)	(PUR, aromatic)	(PUR, aliphatic)	(ESI)	(EP)	(EPC)
Gloss retention	▲	▲	▲	▲	●	■	—	●	●
Colour retention	▲	▲	■	▲	●	■	—	●	●
Resistance to chemica									
Water immersion	▲	■	▲	●	▲	●	▲	■	■
Rain/condensation	■	■	■	▲	■	▲	■	■	■
Solvents	●	●	●	▲	■	▲	■	■	▲
Solvents (splash)	●	●	●	■	■	■	■	■	■
Acids	▲	■	▲	▲	■	▲	●	▲	■
Acids (splash)	■	■	▲	▲	■	■	●	■	■
Alkalis	▲	▲	▲	▲	▲	▲	●	■	■
Alkalis (splash)	■	■	▲	▲	■	■	●	■	■
Resistance to dry heat									
up to 70°C	●	●	▲	■	■	■	■	■	■
70°C to 120°C	—	—	▲	■	■	■	■	■	▲
120°C to 150°C	—	—	▲	●	▲	●	■	▲	▲
>150°C but ≤400°C	—	—	—	—	—	—	■	—	—
Physical properties:									
Abrasion resistance	●	●	●	▲	■	▲	■	■	▲
Impact resistance	▲	▲	▲	▲	■	▲	▲	■	▲
Flexibility	■	■	■	▲	▲	■	●	▲	▲
Hardness	▲	▲	▲	■	■	▲	■	■	■

which the zinc dust pigment content, of the non-volatile portion of the paint, is at least equal to 80% by mass, and (b) other primers (designated as Misc.), are those containing zinc phosphate or other anticorrosive pigments and those in which zinc dust pigment content of the non-volatile portion of the paint is lower than 80% by mass.

In most cases a pre-fabrication primer (shop primer) is applied, just after blast-cleaning, with a thickness of about 15 to 30 μm, to provide temporary anti-corrosion protection during fabrication. The prefabrication primer is overcoated by the specified paint system, with which should be compatible, and it is included in the overall paint, as a further priming coat. The application of the prefabrication primers usually is part, with blast-cleaning, of an automatic procedure, therefore the capacity of the primers for a spray application and a quick drying are essential for their qualification. It is suitable that usual fabrication activities, like welding or gas-cutting, are not significantly impeded by the pre-fabrication primer application, therefore primers are usually certified, not only from safety and health reasons but also with respect to their effect on welding and cutting quality. Normally the pre-fabrication primer is not part of the paint system and if not incorporated, it could be removed.

EN 12944-5, in extended and detailed Tables, includes many alternative types of typical paint systems, appropriate for application, widely used in practice, in which the corrosivity category and the required level of durability are considered. Besides the above parameters, for each type of paint system, the type of binder, the type of primer, as above, the number of coats, the nominal values for the dry film thickness, both for the priming and the subsequent coating, as well as the type of paints, like 1 or 2-pack, are provided. An example of such a Table, for a C3 ambient corrosivity category and low-alloy carbon steel, is presented in Table 8.9. As already mentioned, besides the systems included in the Tables, other types of paint systems could also be used, provided they possess appropriate qualification.

Table 8.9. Paint systems for low-alloy carbon steel for corrosivity category C3 (EN 12944/5)

Substrate: Low-alloy carbon steel
Surface preparation: For Sa $2^{1/2}$, from rust grade A, B or C only (see ISO 8601-1)

System No.	*Priming coat(s)* Binder	Type of primer[a]	No. of coats	NDFT[b] in μm	*Subsequent coat(s)* Binder type	*Paint system* No. of coats	NDFT[b] in μm	*Expected durability* Low	Med	High
A3.01	AK	Misc.	1-2	80	AK	2-3	120	■		
A3.02	AK	Misc.	1-2	80	AK	2-4	160	■	■	
A3.03	AK	Misc.	1-2	80	AK	3-5	200	■	■	■
A3.04	AK	Misc.	1-2	80	AY, PVC, CR[c]	3-5	200	■	■	■
A3.05	AY, PVC, CR[c]	Misc.	1-2	80	AY, PVC, CR[c]	2-4	160	■	■	
A3.06	AY, PVC, CR[c]	Misc.	1-2	80	AY, PVC, CR[c]	3-5	200	■	■	■
A3.07	EP	Misc.	1	80	EP, PUR	2-3	120	■		
A3.08	EP	Misc.	1	80	EP, PUR	2-4	160	■	■	
A3.09	EP	Misc.	1	80	EP, PUR	3-5	200	■	■	■
A3.10	EP, PUR, ESI[d]	Zn (R)	1	60[e]	–	1	60	■	■	
A3.11	EP, PUR, ESI[d]	Zn (R)	1	60[e]	EP, PUR	2	160	■	■	■
A3.12	EP, PUR, ESI[d]	Zn (R)	1	60[e]	AY, PVC, CR[c]	2-3	160	■	■	
A3.13	EP, PUR	Zn (R)	1	60[e]	AY, PVC, CR[c]	3	200	■	■	■

Binder for priming coat(s)	Type	Water-borne possible	Binder for subsequent coat(s)	Type	Water-borne possible
AK = Alkyd	1-pack	X	AK = Alkyd	1-pack	X
CR = Chlorinated rubber	1-pack		CR = Chlorinated rubber	1-pack	
AY = Acrylic	1-pack	X	AY = Acrylic	1-pack	X
PVC = Poly (vinyl chloride)	1-pack		PVC = Poly (vinyl chloride)	1-pack	
EP = Epoxy	2-pack	X	EP = Epoxy	2-pack	X
ESI = Ethyl silicate	1- or 2-pack	X	PUR = Polyurethane, aliphatic	1- or 2-pack	X
PUR = Polyurethane, aromatic or aliphatic	1- or 2-pack	X			

a Zn (R) = Zinc-rich primer.
b NDFT = Nominal dry film thickness.
c It is recommended that compatibility be checked with the paint manufacturer.
d It is recommended for ESI primers that one of the subsequent coats be used as a tie coat.
e It is also possible to work with an NDFT from 40 μm up to 80 μm provided the zinc-rich primer chosen is suitable for such an NDFT

In general, increasing the total dry film thickness and the number of coats, the durability of the paint system is extended. Designing for a corrosivity category more demanding compared to the real conditions, durability is also improved. The number of coats and the nominal dry film thickness of the Tables mentioned above are based on the use of airless spray application. Application by rollers, brushes or conventional spraying equipment will produce a lower film thickness and therefore more coats will be needed to produce the specified total thickness. Information and guidance lines to establish the part of the project specification related to the surface protection is extensively included in EN 12944-8 [8.35].

8.5.5 Design considerations

Besides functional, strength, stability, cost and aesthetic considerations, the durability of a structure is one of the main objectives of the design. In this frame, the overall design should be performed in such a way to facilitate surface preparation, painting operations and maintenance. The shape of the overall structure as well as the arrangement of the individual members and the connections can influence decisively its susceptibility to corrosion. Therefore structures should be designed such that corrosion cannot establish a foothold from which it could be further spread.

Steel components should be designed to be accessible for the purposes of application, inspection and maintenance of the protection paint system. This need is facilitated by fixed walkways, power platforms and similar auxiliary equipment. Components which are at a risk of corrosion or inaccessible after erection, should either be fabricated by a corrosion resistant material or have a protective coating system which remains effective throughout the service life of the structure. Alternatively an allowance for corrosion, by selecting a thicker member could be decided. EN 12944-3 [8.36] deals with design considerations to ensure an efficient protective system for structural members.

Narrow gaps, blind crevices and lap joints are potential points for corrosion attack, arising from retention of moisture and dirt, as well as of abrasives used for the surface preparation. Potential corrosion of this kind, especially in the most corrosive environments, should be avoided by sealing. EN 12944-3 provides suitable typical free distances, required for the use of tools related to corrosion protection purposes, as well as minimum distances between structural members to avoid narrow spaces (see Fig. 8.17). Web stiffeners or similar notches should be ended with a radius not less than 50 mm to allow sufficient preparation and effective application of the paint (Fig. 8.18).

Open box members exposed to surface moisture shall be provided with drain holes or openings. Members with hollow sections exposed to the external conditions should be sealed at their ends by means of continuous welds. In box members and tanks suitable openings should be provided for the safe access of the operators and the equipment. The minimum recommended dimensions for these openings are shown in Fig. 8.19. Supplementary ventilation holes should additionally be designed. For large box sections, the possibility to install dehumidifier equipment, as already done in some bridges, should be examined.

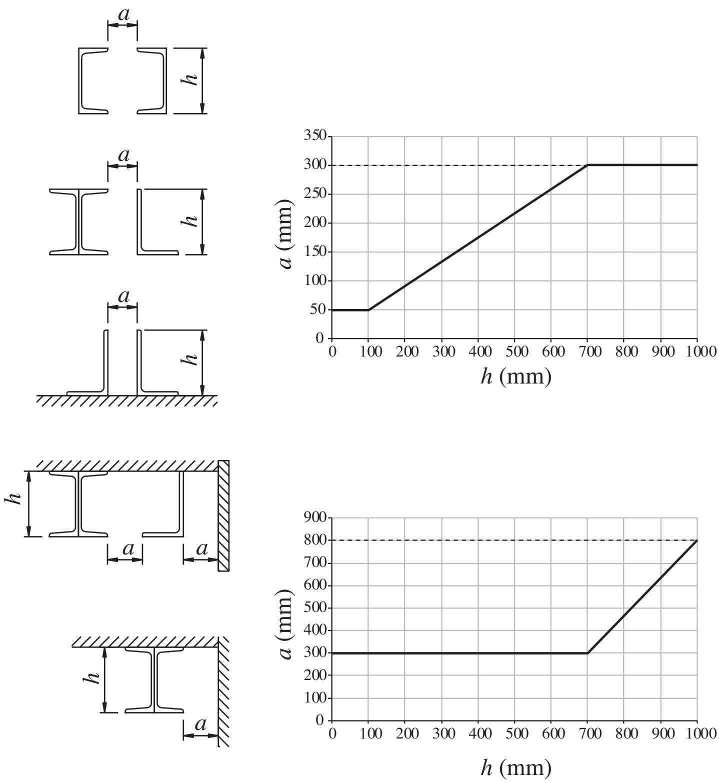


Fig. 8.17. Minimum dimensions for narrow spaces between structural members or between structural members and walls (EN 12944/3)

Rounded edges are desirable to apply uniformly the protective coating with an adequate thickness. Coatings on sharp edges are susceptible to damages during transport and erection, therefore appropriate attention should be given at those edges.

Members should be arranged having such an orientation that the accumulation of deposits or trapped water is avoided (see Fig. 8.20).

From the corrosion protection point of view, welding is preferred against bolting offering a continuous and smooth overall surface. Discontinuous welds should be applied only in connections where the corrosion risk is negligible. Friction surfaces in slip resistant connections should be blast cleaned, prior to assembly, to a minimum preparation grade Sa2 $\frac{1}{2}$ (see 8.5.3) with an appropriate rough-

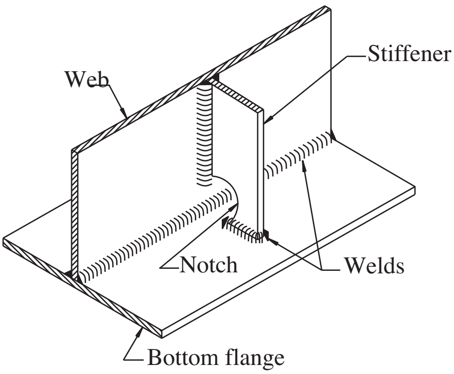


Fig. 8.18. Stiffener design recommended for corrosion protection (EN 12944/3)

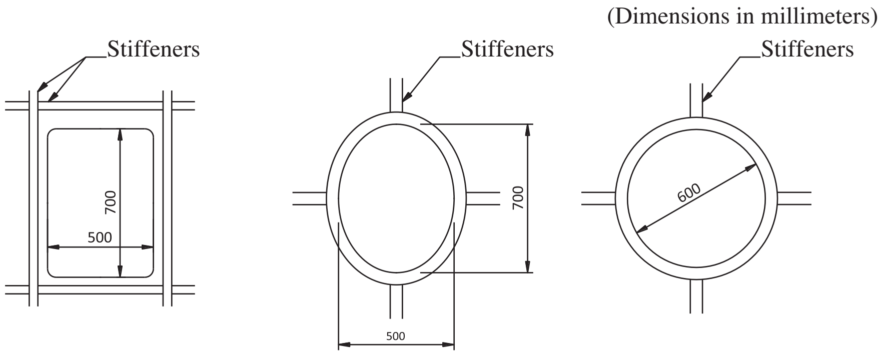


Fig. 8.19. Recommended minimum dimensions of openings for access in box members (EN 12944/3)

ness. A coating material with a suitable friction factor could be applied to the friction surfaces (see 8.4).

Where an electrically conducting joint exists between two metals having different electromechanical potential, in conditions of continuous or repeated exposure to moisture, corrosion of the less noble (more electronegative) metal takes place. The corrosion rate depends mainly on the potential difference between the connected metals, their relative contact areas (unfavorable case small surface of the less noble metal), as well as the nature and the period of the electrolyte action. When this type of joint cannot be avoided, contacting surfaces should be electrically isolated, for example by painting the surfaces of both metals or through isolating washers. Alternatively a cathodic protection could be installed.

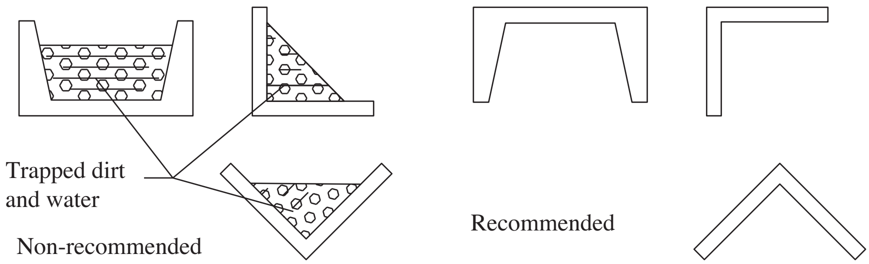


Fig. 8.20. Members arrangement to avoid water and deposits accumulation (EN 12944/3)

8.5.6 Execution and checking of the painting

8.5.6.1 Execution

Painting works can be executed in the workshop, on-site or partially on both. It is better that execution takes place, as far as its main part is concerned, in the workshop where environmental and related conditions are controlled, while on-site secondary painting activities could take place, such as corrections, supplements in welding areas or areas injured during erection. Companies contracted to apply protective paint systems should be experienced in painting works and use qualified personnel. The paint manufacturer's technical data should be respected during all phases of execution, as well as the safety and health of the personnel and the environment protection. The surfaces to be painted, as already mentioned, should be safely and easily accessible and well illuminated. Information on painting execution is provided by EN 12944-7 [8.37].

Coating materials should be stored at temperatures between 3°C and 30°C unless other temperatures are indicated by the manufacturer. The manufacturer should also indicate to the paint container, the date by which the coating expires (shelf life of the paint). Paint coats could be applied by brushes, rollers or by spray. Each coat should be applied, as uniformly as possible, without leaving any areas uncovered. Particular care should be given to surfaces having difficult access. Individual dry film thickness of a coat less than 80% of its nominal value is not acceptable. Thicknesses between 80% and 100% of the nominal value could be accepted, provided that the overall average thickness of the coats is equal or greater to the nominal value. Care should also be taken to avoid areas of excessive thickness. It is recommended that the maximum dry film thickness is not locally greater than three times its nominal value. The lowest and highest temperatures of the coated surface and the surrounding air, accepted for paint application, should be determined in the manufacturer's technical data, in relation to the condensation conditions. In the same technical data sheet, the time interval between successive coats should also be determined.

8.5.6.2 Checking

Checking of the painting can be performed visually and by means of instruments. The uniformity of the paint, its color, the hiding power and probable defects, such as discontinuities, cratering, air bubbles, flaking, cracks or curtains, could be assessed visually. The dry film thickness is measured by appropriate calibrated instruments. The porosity can be checked by flow or high voltage detections while the adhesion to the steel surface can be verified only by applying destructive testing.

The selection of a paint system for a specific application should preferably be based on experience from the application of the system in similar cases, since durability depends on many external factors such as environmental conditions, design arrangements, quality of the surface preparation, care and conditions during application. Laboratory testing is used to assess, compare and select paint systems. In laboratory an artificial accelerated ageing of the paint is produced, reducing the efficiency of the paint protection more rapidly than natural weathering. The artificial conditions are related to a chemical exposure of the specimens, to a water immersion, to an intense condensation or to the exposure to neutral salt spray. The conditions can be uniform during the measures or cyclically applied. The duration of each test varies between 50 and 1500 hours. Information about laboratory testing is provided by EN 12944-7.

8.5.7 Hot dip galvanizing

Paints may protect a structure up to a maximum of 20 to 30 years. A higher protection time, even 100 years, could be provided by metallic protective coatings. The most common type of such a protection is hot dip galvanizing, consisting of a zinc (or zinc alloy) coating applied by immersion of the steel components in a molten bath. The zinc melt contains no more than 2% of other metals. The application of the zinc coating provides an effective method for preventing or retarding corrosion, as the corrosion rate of zinc is much slower than for steel, and the zinc protection works through both barrier and galvanic action. The dimensions of the steel elements to be galvanized are limited by the bath dimensions. In such a way linear components are mainly prefabricated and subsequently spliced by bolting to form a longer member. Information on hot dip galvanizing is given in EN 1461 [8.38] and EN 14713-1 [8.39]. Hot dip galvanizing is very often used in structures erected in marine environments.

The thickness of the protective coating is evaluated through the mean coating thickness and the local minimum one. For steel elements with a thickness greater than 6 mm, the minimum mean value of protection is 85 μm, while the corresponding local minimum value is 70 μm. The mean applied protection thickness can be verified through the mass of the galvanized element, by considering zinc density equal to 7.2 gr/cm^3. The mean thickness of 85 μm corresponds to a mean coating mass, per unit of area, of 610 gr/m^2. A direct determination of the local zinc thickness can be performed by using appropriate instruments, as for instance by measuring the magnetic attraction between a permanent magnet and the base metal, as influenced by the presence of the coating.

The corrosion rate of the zinc depends on the environmental conditions. The zinc loss per year is estimated between 2 and 4 μm, for environments of corrosivity category C4, 1 to 2 μm for environment classified in category C3 and 0.1 to 1 μm for ambient conditions of corrosivity category C2. A surface galvanized by a zinc coating of a minimum thickness of 85 μm has a lifetime, until its first maintenance or repair, between 10 and 20 years in cases of C5 corrosivity conditions, between 20 and 40 years for C4 corrosivity conditions, and between 40 and more than 100 years in environments of corrosivity category C3.

The occurrence of darker or lighter areas (e.g. dark grey areas) or some surface unevenness should not be considered as rejected coatings, provided that the zinc thickness has the required values. Appearance or aesthetic questions should be discussed separately. Adhesion between zinc and base metal is not, in general, tested, as galvanizing is related to adequate contact strength.

At locations where welding will take place after hot dip galvanizing, it is preferable, in prior to the welding process, to remove the coating locally, at a distance at least equal to 10 mm from the affected area. In this way the welding quality is ensured, while after completion of the welding, the galvanic protection should be appropriately, locally restored by thermal spraying or zinc dust paints.

Sometimes for appearance purposes or for a better protection, organic coatings are applied to already galvanized surfaces. When such a protective system is adopted, the term “duplex system” is used to describe the combination of the two coatings.

The total life of a zinc plus organic coating system is greater than the sum of the lives of the individual paint components, as the presence of the zinc layer reduces under-rusting of the organic coating which, in parallel, preserves zinc coating from early corrosion.

A duplex system is often applied in practice, in cases where in zinc coating, after a significant part of its lifetime, an advanced wear is observed and the surface protection should be enhanced, or when zinc coating loses its appearance or becomes degraded. If the additional paint is to be applied before any steel rusting appears, it is better that maintenance works take place when at least 20 to 30 μm zinc thickness remain, as, in this way, longer total life of the protection is ensured.

During preparation of the galvanized surfaces, before application of the organic coating, defective or damaged areas in the zinc surface should be repaired, to restore the protective capacity of zinc coating. Contamination and any foreign material, including marking, should be removed. Zinc coating may be treated by sweep blast-cleaning, using a non-metallic abrasive, to clean the surface or to remove a thin surface layer or a poor adhering coating. After sweep blast cleaning, coating should be continuous and free from adhering or enclosed contaminants that could decrease the durability of both zinc coating and applied paints.

8.5.8 Intumescent coatings

Intumescent coatings are a specific type of paints, offering to the steel structural members a passive protection against fire conditions. Their chemical composition is such that, when temperature increases, they swell, producing an isolating against high temperatures foam. The first patent for this type of paint was submitted in 1938, but their extended use, in cases where a passive protection is required, was developed in the last thirty years.

The thickness of the protecting foam, produced in case of fire conditions, is about 50 or more times the thickness of the initially applied paint, measured in dry conditions. The required isolating foam thickness depends on: (a) the required fire strength, expressed in terms of the minimum time, after fire initiation, during which the structural member should resist, exposed to the relevant conditions, (b) the cross-section characteristics of the member and especially the ratio between the exposed to high temperatures perimeter of the cross-section and the cross-sectional area. This ratio indicates how thin the members of the cross-section are, and therefore how sensitive is the member against a quick temperature increase, (c) the type of fire to which the member is exposed, expressed by a normative time-temperature curve of the surrounding air, and (d) the degree of the member's utilization, as the ratio between the load applied to the member in the fire load combination and the strength of the member in normal conditions. Fire conditions are treated as an accidental load case in EN 1991-1-2 [8.40] (see also section 1.4) and are therefore combined with reduced values of the variable actions, without participation of other accidental loads.

Depending on the aforementioned parameters, the thickness of the intumescent coating varies usually between 400 and 1000 μm. The thickness after drying, which is considered as its nominal value and is introduced in the calculations, is about 70% of the applied paint in wet conditions.

The chemical composition of such paints requires specific knowledge and could change between different manufacturers. However the main materials that the paint should include are: (a) an acid generator (e.g. ammonium polyphosphate) which, when the temperature reaches about 200°C, is decomposed producing an acid (e.g. phosphoric acid), (b) a carbonific material (e.g. pentaerythritol) which, in the range of temperatures between 240°C and 360°C, reacts with the previously mentioned acid, producing an ester and water. The ester, at 360°C, is decomposed giving mainly carbon, as well as water and phosphoric acid, and (c) a blowing agent (e.g. melamine) which, in the aforementioned elevated temperature, produces non-flammable gases. Gases with the already produced, as above, carbon, provide the protective carbon foam.

The steel surfaces, before application of the intumescent coating, should be prepared, usually through a blast-cleaning, and painted by a compatible primer. After application, the intumescent coating is over-coated by one or two protective coats, from an also compatible paint, having a total thickness of about 100 μm, which, in addition to the protective action, gives the color of the overall paint.

Intumescent paints are usually applied by spraying at once, in the shop or on-site, having, for both cases, advantages and disadvantages, as already discussed before. Intumescent paints are a relatively heavy material, therefore, when applied on-site, there is not a significant dispersion in the surrounding area. However it is necessary that protective measures are taken. The in-shop application, in addition to the advantage of an execution under controlled conditions, leads usually to time savings.

8.6 Erection

8.6.1 Introduction

The erection is the last phase of construction, which follows structural design, in-shop fabrication of the separate steel elements, and transportation to the site. During the design appropriate structural systems and constructional details are selected to facilitate productivity and efficiency of fabrication and erection. It is intended that parts of the structures having the largest possible dimensions are prefabricated in the shop to minimize on- site connections.

Foundation bolts are installed during the foundation works before the steel elements are transported to the site. It is important that bolts are positioned with the suitable accuracy to avoid problems, when the erection starts. To this end anchor bolts of the same connection are interconnected by using appropriate thin plates (see Fig. 6.35), which are installed using suitable measuring equipment. Any deviation beyond acceptable limits (see 8.6.4 and 8.7) should be corrected before erection.

8.6.2 Erection method statement

As already mentioned, during all phases of the structural design (conceptual, final and detail design), the designer should select appropriate arrangements, solutions and details, such to facilitate the fabrication of the structural elements as well as their erection on-site. In addition, an efficient and safe erection requires the establishment of a statement describing the erection conditions and environment, the equipment, the methods and the procedure that will be followed for the assembly of the structure in its final position.

An erection statement should include:

a) a description of the site conditions, such as the possibilities of access, the soil description, plans of probable underground services and facilities, overhead electrical cables or other obstacles, as well as information about adjacent buildings or constructions influencing the site activities,
b) the number, the types, the lifting capacities and the radius of operation of the cranes which will be used, as well as the places where these cranes will be positioned with the corresponding soil conditions,
c) the maximum size and weight of the prefabricated or preassembled structural elements to be erected. In the detail design the locations of the on-site connections have already been determined and should be compatible with the above limitations. Otherwise the site connections should be adapted accordingly,
d) the center of gravity of the heavy parts, or parts with an irregular shape,
e) the sequence of erection with a description of the successive steps,
f) the stability concept in all steps of erection, including temporary bracings, restrains or auxiliary structures or cables, which ensure this stability for an appropriate period during erection,
g) the description of any auxiliary structural part of the lifting equipment,
h) the structural verification of all structural members that could be loaded, during erection, in an unfavorable way. For instance, truss bars, which will be submitted mainly in tension, under service conditions of the structure, could become compression elements during erection, depending on the selected lifting nodes of the truss. In such a case buckling failure could result, especially in cases of slender bars,
i) the deformations expected at critical positions of the partially erected structure and probable provisory supporting elements or jacks,
j) concreting activities, in the case of composite structures and clarifications about the lateral support that sheeting could offer to structural elements, and
k) the safety measures to be respected during erection.

The erection statement could be supplemented by a foundation plan showing the arrangement of anchor bolts, the orientation of steel columns, the level of the lower surface of the columns baseplates, the measures to maintain the columns at this level, the method to apply grouting, as well as the methods to protect anchor bolts against damages, especially at their threated part, rust or other materials during erection or concreting.

In many projects the constructor, which is not known during the design elaboration, following his experience, his equipment and the erection techniques with which he is familiar, could propose a safe alternative or a modified erection procedure. This procedure could be applied after an agreement with the owner and the designer.

8.6.3 Marking, handling and storage

All structural components that will be assembled into bigger subsystems, or individually erected should be provisioned with an erection mark. Elements which are preassembled in a sub-structural unit and will be transported and erected as a single element, may have the same initial number as marking. If the member has an orientation (left and right sides) when erected, which is not evident from its shape, this should also be indicated by the marking. The marks should be placed in positions visible during storage and after the erection of the member. Marking should be indicated in the erection drawings, in a way that the place of each element could easily be recognized.

Material delivered to the site and not destined for immediate erection, should be stored in a suitable area, which should be safe against rain water concentrations. Steel elements should be released on wooden sleepers or other suitable materials. I-sections should be stocked so, that water is not gathered in the webs. The same precautions are to be taken for members with hollow sections. Stocking should be organized considering the sequence of the works, so that components to be erected first should be accessible. Fasteners stored on-site should be kept in dry conditions, suitably packed and easy identifiable.

Components should be stocked and handled in such a way that the danger against permanent deformations, or any type of damages to the steel cross-sections or to the protective treatment, is minimized. Damages during transportation, storage or erection should be restored. The restoration procedure for structures classified in execution classes 2, 3 or 4 should be documented.

8.6.4 Anchor bolts and grouting

The inspection of the foundation bolts should be carried out prior to erection, to ensure that they are correctly positioned and securely fixed. Checking is performed by using visual and appropriate measurement means. Any correction should be finished before starting the erection.

In cases where foundation bolts are intended to lightly move inside sleeves (see Fig. 6.36), the sleeves' diameter should be at least three times the bolt diameter and not less than 75 mm. When shims, packings or other supporting steel pieces are used under the baseplates, as temporary supports to ensure the distance between foundation and column's baseplates, they should have flat surfaces, as well as adequate size and strength to avoid local crushing of the foundation concrete. In case where these elements will be incorporated in the grouting, they should be completely covered with a minimum cover of 25 mm.

As far as the required accuracy in the installation of the foundation bolts is concerned, functional tolerances are given by EN 1090-2, the main of which are: (a) for the foundation level, 5 mm more or 15 mm less than the value indicated in the drawings, (b) for an isolated anchor bolt installed in a sleeve or hole, giving the possibility of an adjustment, 10 mm in any horizontal direction and -5 mm to +25 mm for the prescribed protrusion of the bolt. For a group of foundation bolts the tolerance of the horizontal deviation is recommended as 6 mm, (c) for a bolt adhered to the concrete along its whole length, 3 mm for the horizontal deviation, both for an isolated and a group of foundation bolts, and -5 mm to +45 mm for the bolt protrusion.

Before grouting, the space under the baseplate should be carefully cleaned. The space should be completely filled, while vent holes are provided to verify filling. The grouting material should be prepared according to the manufacturer's recommendations. Grouting in temperatures less than 0°C should be avoided. According to EN 1090-2, cement based grout is to be used, with specific characteristics, depending on the thickness of the grouting. Special grouts are also used, with preference to those having low shrinkage performances. When no grouting is to be applied, the perimeter of the baseplate should be sealed using an appropriate material.

Foundation bolts should not be used to secure provisory columns against overturning, unless this is specified in the erection statement and the bolts are verified for the corresponding loading (see also 6.3.6.3).

8.6.5 Erection procedure

The erection procedure should follow the erection method statement, and advance in such a way that the stability of the erected parts is continuously ensured. Every structural part under erection should not be released from the crane before it is connected to the already erected parts, and is secured against lateral instabilities. The parts should also be secured against temporary erection loads, including those due to the erection equipment, and mainly against wind loads. The erection starts from panels of the structure where vertical bracings are provided, either when vertical bracings are placed at the ends or in the middle of the building. This creates an initial stable element that allows to continue the erection procedure.

In single storey portal frames, consisting of linear members (columns and rafters), the parts are usually bolted together on the ground, and the frame is erected in its final position as a whole. In trusses with significant length, the prefabricated parts are also bolted together on the ground, and then erected as a unique element. In case of frames having trusses as rafters, the columns are usually erected first. The truss is carried by the crane (or the cranes) until it is bolted to the waiting columns, and it is connected, at a sufficient number of nodes, to already erected rigid structural parts, in order to ensure its lateral stability, before released from the crane. To this end, a number of purlins and bracings is used. In bolted connections having a significant number of bolts, it is considered in practice, that at least one third of the bolts should be installed, to account for the contribution of the connection to the stability of the partially erected structure.

Special care is to be taken in large structures with significant spans, where the self- weight of the structure could constitute an important part of the total loading, in order to avoid instabilities during erection.

Following the above, two adjacent trusses are frequently erected together with an appropriate number of purlins, transverse bars and bracings (vertical and horizontal) as a rigid box system. An example of such a case is shown in Fig. 8.21. The first and second horizontal trusses will be bolted to the waiting, also boxed, columns.

When lifting heavy I-beams or trusses, erection beams (Fig. 8.22b) are used, in order to avoid the introduction of compressive forces to the erected elements. Otherwise these compressive forces should be considered in the erection statement. Compression forces increase with increased values of the angle α (Fig. 8.22a).

Fig. 8.21. Erection of a pair of plane latticed frames

In structures having main structural elements with significant length, it is optional that initially the two end parts are erected while the central part is lifted in a second step and connected to the already placed ones. Such an example is shown in Fig. 8.23. However, in similar cases, due to the temperature variations and the related expansions and contractions, a gap could be provided during erection between members to be connected, in which specific care should be given. To this end the reference temperature for setting out and measuring the steelwork should be specified in the erection statement.

In multi-storey buildings mainly linear elements are to be transported to the site and erected (columns, main and secondary beams, bracing bars). Cranes able to follow the whole erection process to the full height of the structure are used. It is required that vertical bracing bars should be de-stressed, as erection progresses, to release forces introduced by the vertical loads. This de-stressing should be executed progressively, panel by panel, in order to ensure the overall stability of the structure. Temporary bracings could be used, if needed.

Bolted connections between the separately erected structural parts should not be tightened at their final level until the complete structure, or an important part of it, is definitely aligned and levelled. In this way, the possibility of small movements for the final adjustment is provided. After the exact geometry of the structure is achieved, the bolts could be definitely tightened, as specified.

The alignment and lack of fit in connections may be adjusted using shims. Shims could be welded to either part of the ones to be connected, respecting all limitations recommended by the welding procedures. It is absolutely prohibited that structural parts are forced to be connected. In such a case, forces are introduced to the structural members, and stressing could be increased as additional parts are progressively erected. Accidents during erection are, in many cases, attributed to a forced member

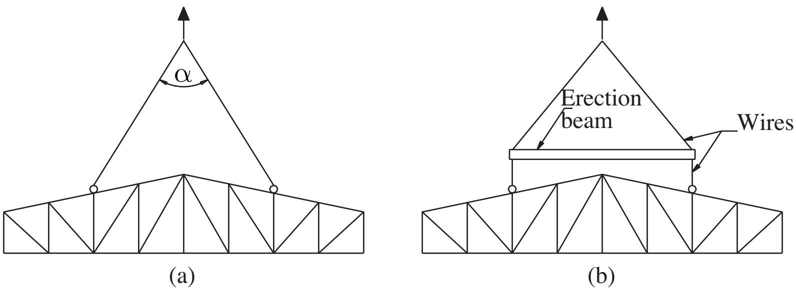


Fig. 8.22. Auxiliary erection beam

Fig. 8.23. Erection of the central part of a large opening structure follows erection of the end parts

adjustment. Any type of difficulty or error, not permitting the unforced connection of the joined parts, should be corrected immediately on –site, before erection continues.

In cases of restricted or limited execution time, a trial erection of the whole structure or of parts of it, could be performed in the shop, before transportation to the site, in order to confirm fit between the fabricated parts or to prove the reliability of some critical operations. A trial erection could also be recommended in cases of distant sites, with difficult access, where correction of possible constructional errors is difficult to be achieved.

8.6.6 Erection tolerances. Survey

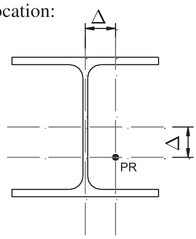
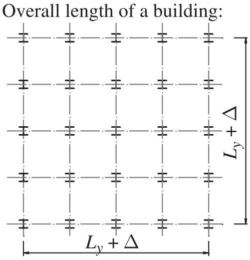
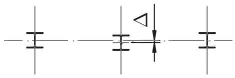
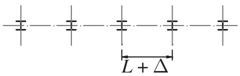
As already mentioned in the introduction, deviations of the structure from the ideal geometry could be accepted, provided that they are sufficiently small to not influence the resistances and stability of the structure (essential tolerances) or create serviceability or functional problems (functional tolerances). Limits of acceptance (tolerances) are given in a normative Annex of EN 1090-2.

The main essential erection tolerances for single storey buildings are:

a) inclination of a single column, measured as the horizontal deviation between the head and the base of the column, equal to h/300 (h the storey height in cm),
b) mean inclination in all columns of the same frame equal to h/500,
c) inclination of a column at the level of the support of a crane supporting beam, equal to h/1000 (h the distance of the runway beam support to the base level), and
d) deviation from the straightness of a single column equal to h/750.

For multi- storey buildings : (a) horizontal deviation of the head of a column in relation to its base, between two successive levels, equal to h/500 (h the storey height), and (b) deviation from the base level of a column head, n levels higher, equal to H/(300n), where H the sum of the heights of all intermediate stories. For beams

Table 8.10. Functional erection tolerances related to the columns positions at their base level (EN 1090-2)

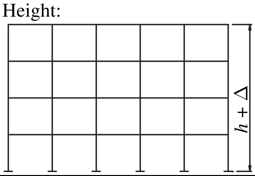
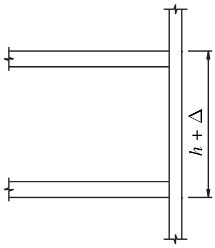
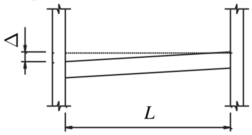
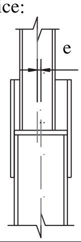
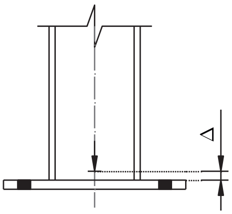
No	*Criterion*	*Parameter*	*Permitted deviation* Δ	
			Class 1	*Class* 2
1	Location: 	Location in plan of the centre of the column at the level of its base, relative to the position point of reference (PR)	Δ = ± 10 mm	Δ = ± 5 mm
2	Overall length of a building: 	Distance between end columns in each line, at base level: $L \leq 30$m $30\text{m} < L < 250$m $L \geq 250$m	Δ = ± 20 mm Δ = ± 0.25(L + 50)mm Δ = ± 0.1(L + 500) mm [L in meters]	Δ = ± 16 mm Δ = ± 0.2(L + 50)mm Δ = ± 0.1(L + 350) mm [L in meters]
3	Column spacing: 	Distance between centres of adjacent columns at base level: L 5m $L > 5$m	Δ = ± 10 mm Δ = ± 0.2 (L+45) mm [L in meters]	Δ = ± 7 mm Δ = ± 0.2 (L+30) mm [L in meters]
4	Column alignment generally: 	Location of the centre of the column at base level, relative to the established column line (ECL)	Δ = ± 10 mm	Δ = ± 7 mm

in general, having a length L, subjected mainly to bending, and components subjected to compression, the tolerance for the deviation from the straightness is equal to L/750.

The functional erection tolerances, recommended by EN 1090-2 and related to the in-plane position of columns, are shown in Table 8.10 for both limitation classes (1 and 2). The corresponding acceptable deviations, along the height of the building, are indicated in Table 8.11. Functional erection tolerances are also given for the inclination of columns for which the essential tolerances are presented above.

As far as erection tolerances related to the beams is concerned, the following limits apply for class 1 limitation (in brackets the corresponding limits for class 2 limitations): (a) for the difference at the level between adjacent beams, measured at corresponding edges, 10 mm (5 mm), (b) for the level of a beam, measured at a beam to column connection 10 mm (5 mm), (c) deviation in the distance between adjacent beams, measured at their ends, 10 mm (5 mm), (d) deviation, in a floor level, of a beam, at a beam to column joint, in relation with the axis of the column, 5 mm (3 mm), and (e) deviation in the straightness of a beam of length L, L/500 (L/1000).

Table 8.11. Functional erection tolerances along the height of a building (EN 1090-2)

No	*Criterion*	*Parameter*	*Permitted deviation* Δ	
			Class 1	*Class* 2
1	Height: 	Overall height, relative to the base level: $h \leq 20$m 20m $< h <$ 100m $h \geq$ 100m	Δ = ± 20 mm Δ = ± 0.5(h+20) mm Δ = ± 0.2(h+200) mm [h in meters]	Δ = ± 10 mm Δ = ± 0.25(h+20) mm Δ = ± 0.1(h+200) mm [h in meters]
2	Storey height: 	Height relative to the adjacent levels	Δ = ± 10 mm	Δ = ± 5 mm
3	Slope: 	Height relative to the other end of a beam	Δ = ± L/500 but $\lvert\Delta\rvert \leq$ 10 mm	Δ = ± L/1000 but $\lvert\Delta\rvert \leq$ 5 mm
4	Column slice: 	Not-intended eccentricity e (about either axis)	5 mm	3 mm
5	Column base: 	Top level of a column base plate relative to its specified level	Δ = ± 5 mm	Δ = ± 5 mm

Functional erection tolerances are also given for the inclination of columns for which the essential tolerances are presented above.

The deviations from the ideal geometry should be checked during erection, through an appropriate system of measurements, using a reference axes' system and selecting representative and characteristic points of the structure to be controlled. The location and frequency for the site measurements should be specified in the inspection plan. The positional accuracy of the erected steelwork is to be measured under the self-weight of the structure only. Details for the site measurements are given in ISO 4463-1 [8.41], while for the methods and the instruments used in ISO 7976-1 [8.42] and ISO 7976-2 [8.43].

8.7 Constructional imperfections

As already mentioned in the introduction, constructional deviations from the ideal geometry of the structural members, due to imperfections created during the in-shop fabrication and the erection, could be accepted in cases they are sufficiently small to not reduce essentially the member's mechanical resistances or to not produce functional problems in the building. The acceptable limits of the above imperfections, named 'tolerances', are included in detail, as already mentioned, in numerous Tables of EN 1090-2, of normative character, covering all usual constructional cases. The deviations are expressed in geometrical terms and measured without including the influence of the self-weight of the member.

It is also already mentioned, that tolerances are divided in two main categories: (a) essential tolerances considered as the imperfection limits for which no essential differentiation is produced concerning the resistances of the members and their stability, and (b) functional tolerances considered as the imperfection limits for which no functionality, appearance or similar problems are created. The same imperfections could be classified as: (a) manufacturing tolerances, related to imperfections produced during the in-shop fabrication and (b) erection tolerances corresponding to imperfections created during the on-site erection of the structural members. In the functional tolerances two different values are given, corresponding to the two classes of limitations, 1 and 2. If one of the two classes is not selected in the execution documents, class 1 is to be applied. Class 2 tolerances are stricter and could be applied for specific parts of the structure for which a better accuracy is required.

Values for essential and functional tolerances are already presented in previous sections concerning holing (8.2.3), anchor bolts (8.6.4) and the deviation of the erected structure from its ideal geometry (8.6.6). A usual case of manufacturing imperfections are related to the welded I-beams, for which the main essential tolerances are included in Table 8.12. In Tables 8.13 and 8.14 functional manufacturing and erection tolerances are presented for the crane supporting beams and the relevant rails, respectively.

In many Tables of EN 1090-2 additional information is provided about: (a) essential and functional manufacturing tolerances concerning cold-formed profiles, flanges of welded cross-sections, welded box sections, web and plate stiffeners, lattice and bracing components, (b) essential or functional, manufacturing or erection tolerances for other, than buildings, structures as bridge decks, towers and masts, conical shells, and (c) functional manufacturing tolerances for individual structural components concerning length, straightness, squareness of the ends, camber.

Table 8.12. Essential manufacturing tolerances for built-up welded I-sections (EN 1090-2)

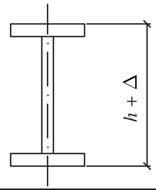
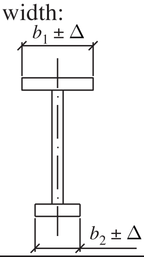
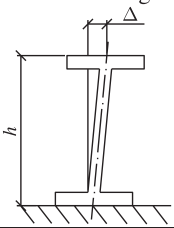
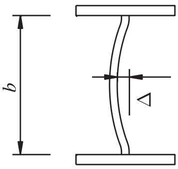
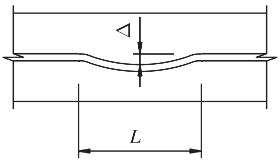
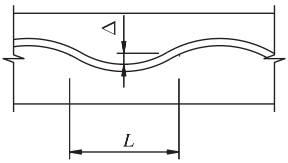
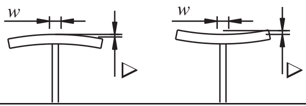
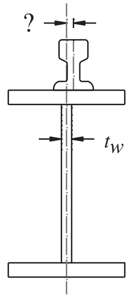
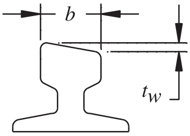
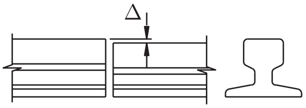
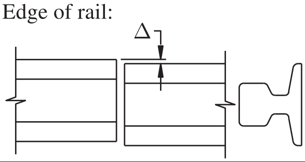
No	*Criterion*	*Parameter*	*Permitted deviation*
1	Depth: 	Overall depth h:	$\Delta = - h/50$ (no positive value given)
2	Flange width: 	Width $b = b_1$ or b_2	$\Delta = - b/100$ (no positive value given)
3	Squareness at bearings: 	Vertically of web at supports, for components without bearing stiffeners:	$\Delta = \pm h/200$ but $\Delta \geq t_w$ (t_w = web thickness)
4	Plate curvature: 	Deviation Δ over plate height b:	$\Delta = \pm b/100$ but $\Delta \geq t$ (t = plate thickness)
5	Web distortion: 	Deviation Δ on gauge length L equal to plate length b:	$\Delta = \pm b/100$ but $\lvert\Delta\rvert \geq t$ (t = plate thickness)
6	Web undulation: 	Deviation Δ on gauge length L equal to plate length b:	$\Delta = \pm b/100$ but $\lvert\Delta\rvert \geq t$ (t = plate thickness)

Table 8.13. Functional manufacturing and erection tolerances for crane beams and rails cross-sections (EN 1090-2)

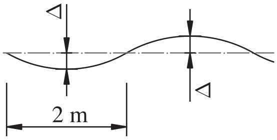
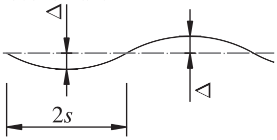
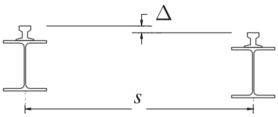
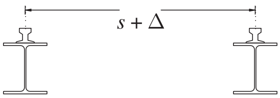
No	*Criterion*	*Parameter*	*Permitted deviation* Δ	
			Class 1	*Class* 2
1	Flatness of top flange of a crane beam: 	Out of flatness over a central width w equal to the rail width plus 10mm either side of rail in nominal position	Δ = ± 1 mm	Δ = ± 1 mm
2	Eccentricity of rail relative to web: 	For $t_w \leq$ 10mm For $t_w >$ 10mm	± 5 mm ± 0.5 t_w	± 5 mm ± 0.5 t_w
3	Slope of rail: 	Slope of top surface of cross-section:	Δ = ± *b*/100	Δ = ± b/100
4	Level of rail: 	Step in top of rail at joint:	Δ = ± 1 mm	Δ = ± 0.5 mm
5	Edge of rail: 	Step in edge of rail at joint:	Δ = ± 1 mm	Δ = ± 0.5 mm

8.8 Quality control

8.8.1 Introduction

Respecting the quality requirements during the construction of a steelwork is the second factor, after an efficient and rational design and of the same importance with it, in order to achieve a reliable and successful technical result. By organizing an efficient quality control system, it is intended that the construction corresponds to the design specifications and requirements, and that corrective actions will take place in case of deviations. To this end, quality control is extended during all phases of the con-

Table 8.14. Functional erection tolerances for the crane runways rails (EN 1090-2)

No	*Criterion*	*Parameter*	*Permitted deviation* Δ *Class* 1	*Permitted deviation* Δ *Class* 2
1	Location of rail in plan:	Relative to the intended location:	Δ = ± 10 mm	Δ = ± 5 mm
2	Local alignment of rail: 	Alignment over 2m gauge length:	Δ = ± 1.5 mm	Δ = ± 1 mm
3	Level of rail	Relative to the intended level:	Δ = ± 15 mm	Δ = ± 10 mm
4	Level of rail	Level over span L of crane beam:	Δ = ± L/500 but |Δ| ≤ 10 mm	Δ = ± L/1000 but |Δ| ≤ 10 mm
5	Level of rail: 	Variation over 2m gauge length:	Δ = ± 3 mm	Δ = ± 2 mm
6	Relative levels of rails on the two sides of a runway: 	Deviation of level: for *s* ≤ 10m for *s* > 10m	Δ = ± 20 mm Δ = ± s/500	Δ = ± 10 mm Δ = ± s/1000
7	Spacing *s* between centres of crane rails: 	Deviation of spacing: for *s* ≤ 16m for *s* > 16m	Δ = ± 10 mm Δ = ± 10 + [*s*-16]/3 mm	Δ = ± 5 mm Δ = ± 5 + [*s*-16]/4 mm
8	Structural end stops:	Relative location of the stops at the same end, measured in the direction of travel on the runway:	Δ = ± *s*/1000 but |Δ| ≤ 10 mm	Δ = ± *s*/1000 but |Δ| ≤ 10 mm

structional works, while procedures to follow control activities should be explicitly determined.

In the previous sections references were presented on the main quality requirements in respect to the connection procedures, such as bolting and welding, the surface protection of the steel structural elements, and the acceptable limits of deviations during fabrication and erection (tolerances). The procedures and actions ensuring compliance with the quality requirements should be determined before construction starts, and be described in a quality plan. All plans, either included in the quality plan or existing as independent guidance texts (welding plan, inspection plan, erection plan), should be elaborated in time and be followed by the personnel, which should be authorized and qualified. In addition, before fabrication starts, the consistency of the constituent products, main and secondary, with the design assumptions should be checked.

8.8.2 Constituent products

8.8.2.1 Mechanical and geometrical characteristics

Constituent products used for the execution of steel structures, such as structural steel elements, bolt assemblies or welding consumables, should correspond to relevant European Standards. If not, or for supplementary and secondary structural parts, the mechanical and other specific material properties should be appropriately specified. In all cases products should be, when delivered, accompanied by appropriate certifications, proving that the supplied products correspond to those ordered, and ensuring compatibility with the requirements for material properties and product standards. The product standards for hot-rolled cross-sections (I and H-sections, channels, equal or unequal leg angles, T-sections, plates), from usual carbon steel, are provided by EN 10025-1 [8.44], as already mentioned in chapter 1, while for hot-finished and cold-formed hollow sections by EN 10210-1 [8.45] and 10219-1 [8.46], respectively. The types of certifications delivered with the corresponding products are specified in EN 10021 [8.47].

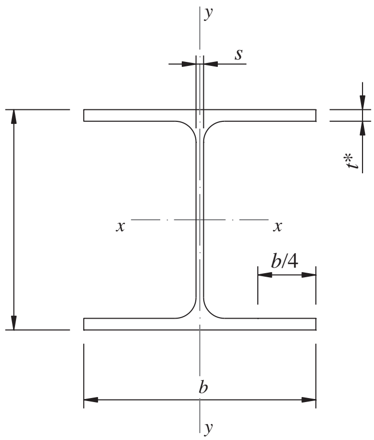
For steel used in the fabrication of structural members, a minimum ductility is required. The following limitations, for the steel characteristic properties (see 1.5.2), are recommended by EN 1993-1-1 [8.48]: (a) ratio between ultimate and yield stresses greater than 1.10, (b) elongation at the failure not less than 15%, and (c) ratio between ultimate and yield strains greater than 15. Steel grades presented in Table 1.11 fulfill the above limitations. National Annexes could define different ductility requirements, for instance in areas of important seismicity.

When it is required that a certification is related to testing, the test document should include: the client's name and order number, the type of cross-section, the steel grade, the batch from which the elements are produced, the chemical composition, the mechanical characteristics (tensile strength, yield stress, ultimate strain, material toughness at -20°C). In the certificate should also be indicated if testing is related to the overall production process, applied by the manufacturer, or to a specific batch from which those elements are originate.

When constituent products of different steel grades or qualities are used in the same structure, each item should be designated with a mark identifying its grade, except for structures of EXC1. For structures belonging to EXC3 or EXC4, all constituent products should be traceable at all phases of construction, from the receipt to the handover.

The tolerances on the cross-sectional shape and the overall dimensions of the structural steel elements are provided by specific norms. As an example, Table 8.15 presents the limits of acceptable deviations of the cross-sectional dimensions (total height, flanges width, web and flanges thickness) for I and H cross-sections (EN 10034 [8.49]). In addition, the tolerance for the web eccentricity, in relation to the flanges, varies between 2.5 and 5 mm depending on the flange width, while for the out-of-square deviation the tolerance is equal to 2% of the flange width and no more

Table 8.15. Dimensional tolerances for structural steel I and H sections (EN 10034)



*t** is measured in *b*/4

Section height h		*Flange width b*		*Web thickness s*		*Flange thickness t*	
height mm	tolerance mm	width mm	tolerance mm	Thickness mm	tolerance mm	thickness mm	tolerance mm
$h \leq 180$	+3.0 -2.0	b ≤ 110	+4.0 -1.0	$s < 7$	±0.7	$t < 6.5$	+1.5 -0.5
180 < h ≤ 400	+4.0 -2.0	110 < b ≤ 210	+4.0 -2.0	$7 \leq s < 10$	±1.0	$6.5 \leq t < 10$	+2.0 -1.0
400 < h ≤ 700	+5.0 -3.0	210 < b ≤ 325	+4.0 -4.0	$10 \leq s < 20$	±1.5	10 ≤ t < 20	+2.5 -1.5
$h > 700$	+5.0 -5.0	b > 325	+6.0 -5.0	$20 \leq s < 40$	±2.0	$20 \leq t < 30$	+2.5 -2.0
				$40 \leq s < 60$	±2.5	$30 \leq t < 40$	+2.5 -2.5
				$s \geq 60$	±3.0	$40 \leq t < 60$	+3.0 -3.0
						$t \geq 60$	+4.0 -4.0

than 6.5 mm. The length of the element should not deviate by more than 50 mm from the dimension ordered, while the deviation from the straightness, in both main axes, should be no more than 0.3%, 0.15% and 0.1% of the element length, when the cross-section height is less 180 mm, between 180 and 360 mm or larger than 360 mm, respectively. Finally, the tolerance on the element mass is 4%, with reference mass the one corresponding to a steel density of 7.85 gr/cm^3.

The corresponding specifications for other types of cross-sections are mentioned in EN 1090-2. Tolerances in regard to the thickness of plates are provided by EN 10029 [8.50], depending on their nominal thickness values. For thicknesses of a single plate between 8 and 15 mm, the limits of acceptable deviations are -0.5 mm and +1.2 mm, while for thicknesses between 25 and 40 mm the above values are -0.8 and +1.4 mm, respectively. For plates in structures of EXC4, a fixed minus tolerance of 0.3 mm is recommended. Limitations are finally provided regarding the differences in thickness within the same plate.

8.8.2.2 Surface conditions

Besides the mechanical and geometrical characteristics of the constituent products, the surface conditions of the delivered material should also be inspected before use. EN 10163 deals with it, provides limits for steel surface discontinuities and recommends methods of repair, if needed. In its part 1 [8.51] the frequent discontinuities are listed as, for example, (a) blow holes (blisters), located closely beneath the surface which often appear during hot-rolling, (b) non-metallic inclusions, (c) marks in the rolled surfaces, (d) scratches or grooves, (e) cracks, in the form of narrow surface fracture lines due mainly to material stresses which often develop during cooling, (f) depressions or protuberances, (g) seams, caused mainly when small imperfections in the semi-product are elongated and extended during rolling, and (h) overlapping material, partially connected to the base one.

Part 2 of EN 10163 [8.52] is related to checking of the delivered plates. For each range of plate thicknesses, tolerances for the discontinuities, as above, are provided. In this norm plates are divided in two classes, A and B, and each class in three subclasses 1, 2 and 3. Subclasses are related to the repair of the plates. Repair could be performed by grinding or welding. In buildings the rules of class A2 is recommended to apply. Subclass 2 means that repair by welding is only permitted if agreed at the time of the order and under agreed conditions. As an example, the recommended discontinuity limitations for plates of thickness between 8 and 25 mm are presented:

a) for isolated discontinuities, other than cracks, shell or seams, a maximum depth of 0.3 mm is acceptable, independent of their number.
b) many discontinuities, of depth less than 0.3 mm, are acceptable when they don't cover more than 15% of the inspected surface.
c) discontinuities, other than cracks, shell or seams, with a depth more than 0.3 mm, but less than 0.5 mm, where the sum of the affected areas does not exceed the 5% of the inspected surface, are tolerated. The affected area of a discontinuity is the surface included by a line distant 20 mm along the perimeter of the discontinuity.
d) it is accepted that the plate thickness under the discontinuities is less than the minimum accepted value specified by EN 10029 [8.50], provided that this appears in less than 2% of the inspected surface.
e) discontinuities with a depth exceeding 0.5 mm should be repaired.
f) discontinuities with a depth between 0.3 and 0.5 mm, having a total affected area more than 5% of the inspected surface should be repaired.
g) discontinuities as cracks, shell or seams, should be repaired irrespective to their depth and number.

When elements from usual cross-sections are delivered, the rules of the Part 3 of EN 10163 [8.53] apply, in respect to the surface inspection. Steel elements are divided in two classes, C and D, while the same, as for the plates, subclasses, concerning repair requirements, are used. For buildings, classification C1 is recommended. Subclass 1 means that repair by chipping or grinding followed by welding is permitted under some conditions. Tables providing tolerances for the surface discontinuities, depending on the product thickness, are included in this part. As an example, for products with thickness between 6 and 20 mm, the tolerance for the maximum discontinuity depth is 1.2 mm, while for thicknesses between 20 and 40 mm it is 1.7

mm. Discontinuities with a surface area, in which the remaining thickness under the discontinuity is less than the minimum required value by EN 10029, larger than 15% of the inspected surface, should be repaired.

For hollow sections where often surface dents are presented, especially in cases of thin walls, the inspection could be performed as shown in Fig. 8.24. The straight length L should be at least equal to 2d, the tolerance for the gap Δ is d/100 and, in any case, not more than 2mm. If the gap exceeds the previous limits, the repair could be carried out by welding of local cover plates of the same, as the product, thickness.

Finally, when steel products are ordered with additional specific properties, the inspection should be accordingly supplemented.

8.8.3 Quality procedures

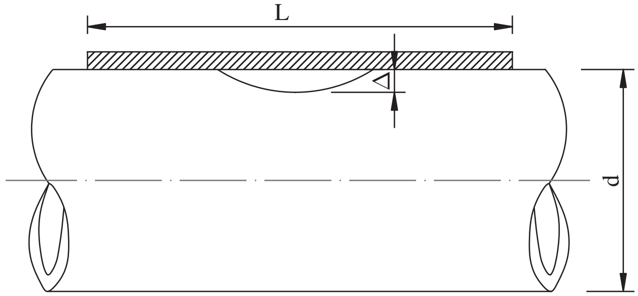
For the execution of a structure in the anticipated quality, care should be given, during all fabrication and erection phases, for the respect of the specifications, quality requirements and contractual texts, following a detailed and explicit plan. It is intended that any defect or deviation from the specifications or the required quality level is ascertained and repaired, if needed, the earlier possible, so that the cost of the corrections, as well as the overall cost of the project or the delays in the execution of the works, are minimized.

The owner is usually establishing, in the frame of the existing Codes, the project specification, through the design team. The project specification may establish: (a) the method and procedure of the erection, including all provisory elements ensuring stability of the structure in all intermediate phases, (b) the position and type of the on-site executed connections, (c) the reference temperature, (d) the tolerances on the constructional and geometrical imperfections, (e) the schedule of inspections during construction, and (f) the qualification criteria for the steelwork contractor and any subcontractors.

The fabricator should submit to the owner an execution specification and a quality plan, which should also be agreed between the parts involved, before the start of the works. In these documents, pending or missing items could be clarified and agreed. Alternative execution specifications, compared to the included in the project specification, could also examined and adopted. The subject of the execution specification is usually: (a) the clarification of the execution class in which the structure is classified, if this class has not been already determined, (b) the tolerances' class, (c) the technical requirements concerning the safety of the works, (d) the preparation grade related to the corrosion protection, (e) additional information or options concerning constituent products, not covered by listed standards or the drawings, preparation of the structural parts, assembly, welding, mechanical fastening, erection, inspection testing and corrections, corrosion protection, and (f) a quality documentation, if not covered by a quality plan as below. This documentation is recommended for projects of execution classes 2, 3 or 4.

A check list concerning the items which a quality plan should contain is included in an Annex of EN 1090-2. The main parts of such a plan are related to:

a) the management of the project and, more specifically, the project management organization, including functions and responsibilities, the relations with subcon-



$L \geqslant 2d,\ \Delta \leqslant \max(d/100,\ 2mm)$

Fig. 8.24. Inspection of delivered hollow sections against surface cleats

tractors and other third parties, the identification of qualified personnel to be employed in the project, such as the welding coordination and inspection personnel, welders and welding operators,

b) a review of the documentation, prior to the execution, determining the necessary certificates to be received for the constituent products and the consumables, when delivered, the welding procedure specifications and the relevant qualification records, the erection methods and the justification of temporary works during erection, the methods of preloading fasteners, the arrangement for third parties approvals, if any,
c) the procedures for incorporating revisions during execution, in order to avoid the use of invalid documents in house or by the subcontractors,
d) the execution records concerning inspection, test reports and actions, when nonconformities are presented, related to the preparation of joint faces prior to welding, the welding, the geometrical tolerances of the manufactured components, the surface preparation and the calibration of the equipment, including the one for preloading of the bolts,
e) the execution records relevant to the erection, such as the delivery sequence to the site of successive schedules, the dimensional survey, actions to be taken when nonconformities appear, the certification for the completion of the erection and its handover,
f) the acceptance criteria, the release and rejection procedures for fabrication and erection activities,
g) the specific requirements and additional procedures, in cases of higher execution classes,
h) the required documentation records, after completion of the project works.

Quality requirements for specific constructional parts, such as the connecting means, welding and bolting, are extensively presented in 8.3 and 8.4, respectively. Quality requirements for the corrosion protection are presented in 8.5 while the significance of a reliable, safe and efficient erection is explained in 8.6. Specific plans, like the welding plan (see 8.3.2), the inspection and testing plans as well as the erection plan, which are necessary for the qualitative execution of the work, can exist as separate documents or being incorporated in the general documents of the project and the execution specifications or in the quality plan.

After the end of the works the steelwork constructor should prepare the execution documentation, such as a record of the as-built structure, including all information proving that the execution was performed as specified. In this document the following should be included: (a) the constituent product certificates, (b) the results of the quality tests and verifications performed during construction, certifying that the required quality is achieved, (c) the measurements related to the final geometry of the structure, and (d) the description of any deviation from the project specification or other contractual documents and the corrective actions undertaken. The execution documents should remain available for a period of at least equal to five years, except if a longer period is agreed or required.

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9
Design Examples

Abstract. This chapter presents fifty-two representative numerical examples, based on the design rules for the verification of cross-sections and members, subjected to the usual types of loading, the verification of bolted and welded connections, as well as for specific items such as hollow section joints, uniform built-up compression members or column bases. The calculation steps are directly related in the text with the corresponding numbers of paragraphs, equations, tables or figures of the Eurocodes, which are highlighted in grey. If only numbers appear, it is understood that reference is made to the part of Eurocode 3 mentioned in the beginning of each example. Otherwise, the specific Eurocode is additionally mentioned.

9.1 Example: Combination of actions

The main frames of a steel building consist of moment resisting frames (Fig. 9.1) at distances of 5 m between them. Each frame is subjected to following loads: permanent loads $g = 0.20$ kN/m$^{2\cdot}$ snow load $s = 0.75$ kN/m$^{2\cdot}$ wind action $w = 0.50$ kN/m$^{2\cdot}$ live load on the, non-accessible, roof $q = 0.40$ kN/m^2 and seismic coefficient $\varepsilon = 0.15$ (ratio between horizontal seismic forces and vertical loads in the seismic situation). To be determined are the design moments at the nodes B and D for the ultimate limit state.

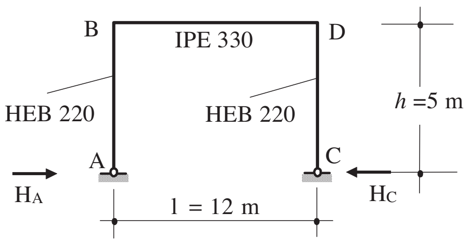


Fig. 9.1. Geometry and cross-sections of the frame

I. Vayas et al., *Design of Steel Structures to Eurocodes*, Springer Tracts in Civil Engineering, https://doi.org/10.1007/978-3-319-95474-5_9

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.1.1 Loads and imperfections

Permanent loads	$g = 0.20 \cdot 5 = 1.0$ kN/m
Snow	$s = 0.75 \cdot 5 = 3.75$ kN/m
Wind	$w = 0.50 \cdot 5 = 2.5$ kN/m
Live	$q = 0.40 \cdot 5 = 2.0$ kN/m

The following coefficients for the wind action will be considered: pressure coefficient $c_p = 1.3$ and lift coefficient $c'_p = 0.6$.

As a simplification, the wind pressure will be entirely applied windward only. The basic value for the imperfection is: $\phi_0 = \frac{1}{200}$. 5.3.2(3a)

$$\alpha_h = \frac{2}{\sqrt{5}} = 0.894 \quad \text{with} \quad 2/3 < \alpha_h < 1$$

$$\alpha_m = \sqrt{0.5 \cdot (1 + 1/2)} = 0.866$$ Eq. 5.5

$$\text{so,} \quad \phi = \frac{1}{200} \cdot 0.894 \cdot 0.866 = \frac{1}{258}$$ Eq.5.8

It can be easily shown that for the columns, the following formula is valid:

$$\bar{\lambda} < 0.5 \cdot \sqrt{\frac{A \cdot f_y}{N_{Ed}}}$$

and therefore, local bow imperfections need not be taken into account.

9.1.2 Frame analysis

$$k = \frac{I_b}{I_c} \frac{h}{l} = \frac{11770}{8090} \cdot \frac{5}{12} = 0.61$$

9.1.2.1 Permanent load G

Total vertical force: $N_G = 1.0 \cdot 12 = 12$ kN.

Instead of the initial sway imperfection, an equivalent horizontal force is considered:

$$H = \phi \cdot N = \frac{1}{258} \cdot 12 = 0.05 \text{ kN}$$ Fig. 5.4

The loading and the corresponding bending moments due to this load are shown in Fig. 9.2.

$$H_A = \frac{gl^2}{4h(2k+3)} - \frac{H}{2} = \frac{1 \cdot 12^2}{4 \cdot 5 \cdot (2 \cdot 0.61 + 3)} - \frac{0.05}{2} = 1.71 - 0.03 = 1.68 \text{ kN}$$

$$H_C = 1.71 + 0.03 = 1.74 \text{ kN}$$

$$M_B = -1.68 \cdot 5 = -8.4 \text{ kNm}$$

$$M_D = -1.74 \cdot 5 = -8.7 \text{ kNm}$$

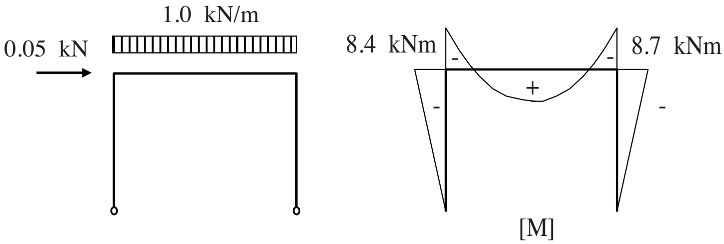


Fig. 9.2. Loading and bending moments due to permanent load

9.1.2.2 Snow S

Total vertical force: $N_S = 3.75 \cdot 12 = 45$ kN.

Equivalent horizontal force due to imperfections:

$$H = \phi N = \frac{1}{258} \cdot 45 = 0.174\text{kN}$$

The loading and the corresponding bending moments due to snow load are shown in Fig. 9.3

$$H_A = \frac{sl^2}{4h(2k+3)} - \frac{H}{2} = \frac{3.75 \cdot 12^2}{4 \cdot 5 \cdot (2 \cdot 0.61 + 3)} - \frac{0.174}{2} = 6.40 - 0.09 = 6.31 \text{ kN}$$
$$H_C = 6.40 + 0.09 = 6.49 \text{ kN}$$
$$M_B = -6.31 \cdot 5 = -31.6 \text{ kNm}$$
$$M_D = -6.49 \cdot 5 = -32.5 \text{ kNm}$$

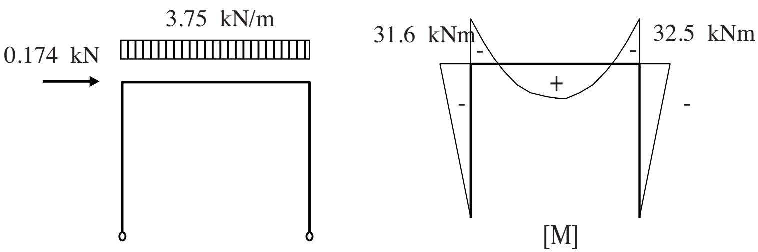


Fig. 9.3. Loading and bending moments due to snow load

9.1.2.3 Wind W

Since wind is to be combined with all actions for which the frame has an initial sway imperfection, the same imperfection should be considered also for wind.

Lateral force: $w_h = 1.3 \cdot 2.5 = 3.25$ kN/m;

Vertical force: $w_v = 0.6 \cdot 2.5 = 1.5$ kN/m;

Total vertical force: $N_w = 1.5 \cdot 12 = 18$ kN.

Instead of the initial sway imperfection, an equivalent horizontal force is considered:

$$H = \phi \cdot N = \frac{1}{258} \cdot 18 = 0.07\ \text{kN}$$

The loading and the corresponding bending moments due to the wind actions are shown in Fig. 9.4.

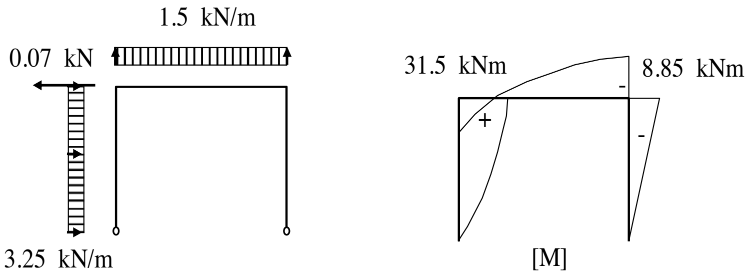


Fig. 9.4. Loading and bending moments due to wind actions

$$H_A = \frac{H}{2} - \frac{w_v l^2}{4h(2k+3)} - \frac{w_h h}{8} \frac{11k+18}{2k+3} =$$

$$= \frac{0.07}{2} - \frac{1.5 \cdot 12^2}{4 \cdot 5 \cdot (2 \cdot 0.61 + 3)} - \frac{3.25 \cdot 5}{8} \cdot \frac{11 \cdot 0.61 + 18}{2 \cdot 0.61 + 3} =$$

$$= 0.035 - 2.56 - 11.89 = -14.42\ \text{kN}$$

$$H_C = -\frac{H}{2} - \frac{w_v l^2}{4h(2k+3)} + \frac{w_h h}{8} \frac{5k+6}{2k+3} =$$

$$= -\frac{0.07}{2} - 2.56 + \frac{3.25 \cdot 5}{8} \cdot \frac{5 \cdot 0.61 + 6}{2 \cdot 0.61 + 3} = -0.035 - 2.56 + 4.36 = 1.77 kN$$

$$M_B = 14.42 \cdot 5 - 3.25 \cdot \frac{5^2}{2} = 31.5\ \text{kNm}$$

$$M_D = -1.77 \cdot 5 = -8.85\ \text{kNm}$$

9.1.2.4 Live load on the roof Q

Total vertical force: $N_Q = 2 \cdot 12 = 24$ kN

Instead of the initial sway imperfection, an equivalent horizontal force is considered:

$$H = \phi N = \frac{1}{258} \cdot 24 = 0.093\ \text{kN}$$

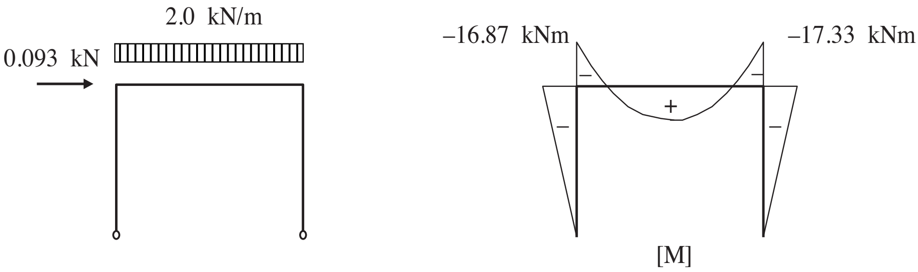


Fig. 9.5. Loading and bending moments due to variable load

9.1.2.5 Earthquake

Total vertical force:

$$N = N_G + \sum \psi_{2i} Q_i =$$
$$= N_G + 0.20 N_S + 0 N_Q =$$
$$= 12 + 0.20 \cdot 45 + 0 \cdot 24 = 21.0\,\text{kN}$$

EN 1998-1. Eq. 3.17

in which the combination factor for the snow is taken as $\psi_2 = 0.20$, for the live loads (non accessible roof) and the wind $\psi_2 = 0$

Horizontal seismic force

$$E = 0.15N = 0.15 \cdot 21.0 = 3.15\,\text{kN}$$

In this case sway imperfection may be disregarded since $\varepsilon = \frac{H_{Ed}}{V_{Ed}} = 0.15$ and Eq. 5.7 of EN 1993-1-1 is satisfied.

Loading and bending moments due to seismic load see Fig. 9.6.

$$-H_A = H_C = 3.15/2 = 1.575\,\text{kN}$$
$$M_B = -M_D = 1.575 \cdot 5 = 7.87\,\text{kN}$$

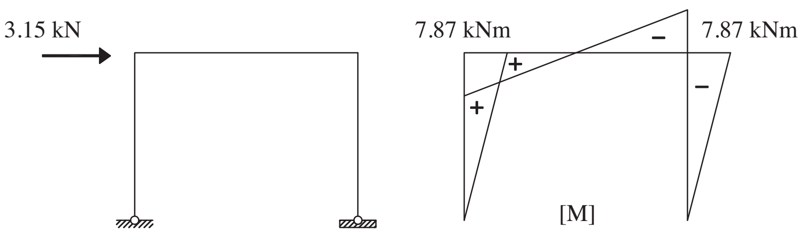


Fig. 9.6. Loading and bending moments due to seismic load

9.1.3 Combination of actions

9.1.3.1 Design bending moment at B

The above partial actions lead to the following moments:

$$M_G = -8.4 \text{ kNm}$$
$$M_S = -31.6 \text{ kNm}$$
$$M_W = 31.5 \text{ kNm}$$
$$M_Q = -16.87 \text{ kNm}$$
$$M_E = 7.87 \text{ kNm}$$

Basic combination

The following combinations will be examined:

$$\gamma_G G + \gamma_{Q1} Q_1 + \psi_0 \sum \gamma_{Qi} Q_i$$

EN 1990. Eq. 6.10

a. *S* as main action

$$M_{Ed} = 1.35 \cdot (-8.4) + 1.5 \cdot (-31.6) + 0 \cdot 1.5 \cdot (-16.87) = -58.7 \text{ kNm}$$

b. *Q* as main action

$$M_{Ed} = 1.35 \cdot (-8.4) + 0.7 \cdot 1.5 \cdot (-31.6) + 1.5 \cdot (-16.87) = -69.8 \text{ kNm}$$

c. *W* as main action

$$M_{Ed} = 1.0 \cdot (-8.4) + 1.5 \cdot 31.5 = 38.8 \text{ kNm}$$

In the above, the ψ_0 factors are taken from the relevant Annex A1 (EN 1990. Table A.1.1) as follows (see Table 1.4):

$$Q : \psi_0 = \psi_2 = 0 \qquad S = \psi_0 = 0.7, \psi_1 = 0.2 \qquad W = \psi_0 = 0.6, \psi_2 = 0$$

Earthquake combination

$$M_{Ed} = -8.4 + 0.20 \cdot (-31.6) + 7.87 = -6.85 \text{ kNm}$$

(The moments due to the variable loads lead to favorable results so they are not considered).

Final design moments at B:

$$\min M_{Ed} = -69.8 \text{ kNm}$$
$$\max M_{Ed} = 38.8 \text{ kNm}$$

9.1.3.2 Design bending moment at D

The partial actions lead to the following moments:

$$M_G = -8.7 \text{ kNm}$$
$$M_S = -32.5 \text{ kNm}$$
$$M_W = -8.85 \text{ kNm}$$
$$M_Q = -17.33 \text{ kNm}$$
$$M_E = -7.87 \text{ kNm}$$

Using the same procedure as in B:

Basic combination

$$M_{Ed} = 1.35 \cdot (-8.7) + 1.5 \cdot (-32.5) + 0.6 \cdot 1.5 \cdot (-8.85) + 0 \cdot 1.5 \cdot (-17.33) = \\ = -68.5 \text{ kNm}$$
$$M_{Ed} = 1.35 \cdot (-8.7) + 1.5 \cdot 0.7 \cdot (-32.5) + 0.6 \cdot 1.5 \cdot (-8.85) + 1.5 \cdot (-17.33) = \\ = -79.8 \text{ kNm}$$
$$M_{Ed} = 1.35 \cdot (-8.7) + 1.5 \cdot 0.7 \cdot (-32.5) + 1.5 \cdot (-8.85) + 0 \cdot 1.5 \cdot (-17.33) = \\ = -59.10 \text{ kNm}$$

Earthquake combination

$$M_{Ed} = -8.7 - 0.20 \cdot 32.5 - 0 \cdot 8.85 - 0 \cdot 17.33 - 7.87 = -23.1 \text{ kNm}$$

Final design moment at D:

$$\min M_{Ed} = -79.8 \text{ kNm}$$

Remark 1. Although the wind and earthquake actions may change sign and apply in ($\pm x$) directions, in this example they were examined only in the $+x$ direction. This is consistent with the direction of the equivalent horizontal force due to the initial frame inclination for the loadcases G, S and Q.

9.2 Example: Classification of an (I) cross-section

An IPE 500 (grade S 275) cross-section is to be classified for the following cases:

1. pure compression,
2. pure bending, and
3. bending combined with an axial compressive force equal to 30% of the cross-section plastic strength due to normal forces.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

$$\text{Steel S275. } f_y = 275\text{N/mm}^2 \text{ and } \varepsilon = \sqrt{\frac{235}{275}} = 0.92$$ Tab. 5.2

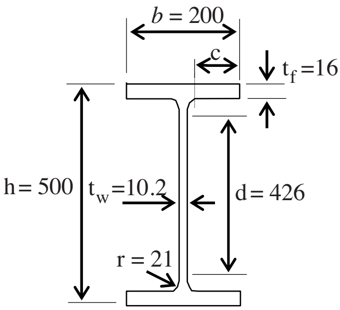


Fig. 9.7. Geometry of the IPE 500 cross-section

9.2.1 Pure compression

Web: Tab. 5.2. sheet 1

$$c = d = 426\text{ mm},\quad t = 10.2\text{ mm}$$

$$\frac{c}{t} = \frac{426}{10.2} = 41.7 < 42\varepsilon = 42\cdot 0.92 = 38.64$$

The web belongs to class 4.

Flange: Tab. 5.2. sheet 2

$$c = \frac{200-10.2}{2} - 21 = 73.9,\quad t = 16$$

$$\frac{c}{t} = \frac{73.9}{16} = 4.62 < 9\varepsilon = 9\cdot 0.92 = 8.3$$

The flange belongs to class 1.

So, in this case the cross-section belongs to class 4. 5.5.2(6)

9.2.2 Pure bending

Web: Tab. 5.2. sheet 2

$$\frac{c}{t} = \frac{426}{10.2} = 41.7 < 72\varepsilon = 72\cdot 0.92 = 66.20$$

The web belongs to class 1.

The flange belongs to class 1 (as in case 1).

So, in this case the cross-section belongs to class 1. 5.5.2(6)

9.2.3 Compression and bending

Due to the axial force N = 0.30 N_{pl} uniform stresses equal to 0.30 f_y develop in the cross-section, which are added to the stresses that result from bending.

Web: Tab. 5.2. sheet 1

1. In case of an elastic stress distribution (see Fig. 9.8)

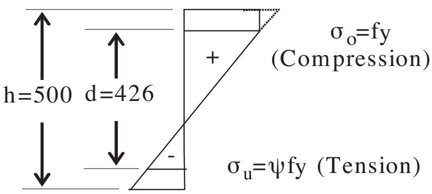


Fig. 9.8. Elastic stress distribution

The normal stress is: $\sigma = \frac{\mathrm{N}}{\mathrm{A}} + \frac{\mathrm{M}}{I} z$
Upper point of the web:

$$\sigma_o = 0.3 f_y + \frac{M}{I} z = f_y \Rightarrow \frac{M}{I} z = 0.7 f_y$$

Lower point of the web:

$$\sigma_u = 0.3 f_y - \frac{M}{I} z = 0.3 f_y - 0.7 f_y = -0.4 f_y$$

So $\psi = \frac{\sigma_u}{\sigma_o} = \frac{-0.4 f_y}{f_y} = -0.4 > -1$

$$\frac{c}{t} = \frac{426}{10.2} = 41.7 < \frac{42\varepsilon}{0.67 + 0.33\psi} = \frac{42 \cdot 0.92}{0.67 - 0.33 \cdot 0.4} = 71.8$$

and the web belongs to class 3 or lower.

It should be examined if the web belongs to class 1 or 2. assuming plastic distribution of stresses (Fig. 9.9).

The applied axial force $N = 0.30\ N_{pl}$ is equal to the difference between the compression and the tension force that appear in the upper and lower part of the cross-section in relation to neutral axis.

Compression force:

$$D = (A_f + \alpha d t_w) f_y$$

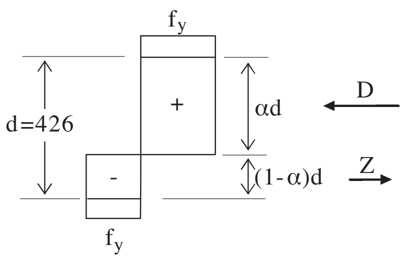


Fig. 9.9. Plastic distribution of stresses

Tension force:

$$Z = (A_f + (1-\alpha)dt_w)f_y$$

Applied axial force:

$$N = D - Z \quad \text{or} \quad 0.30Af_y = (A_f + \alpha dt_w)f_y - (A_f + (1-\alpha)dt_w)f_y \quad \text{or}$$

$$0.30A = (2\alpha - 1)dt_w \quad \text{or}$$

$$a = \frac{1}{2}\left(\frac{0.30\text{A}}{dt_w} + 1\right) = \frac{1}{2}\left(\frac{0.30 \cdot 116}{42.6 \cdot 1.02} + 1\right) = 0.90$$

Symbols:

$$A_f = \text{area of the flange} \qquad A = \text{total area}$$

Check if the web belongs to class 1.

$$\alpha > 0.5 \Rightarrow \frac{c}{t} = \frac{426}{10.2} = 41.7 \leqslant \frac{396\varepsilon}{13\alpha - 1} = \frac{396 \cdot 0.92}{13 \cdot 0.90 - 1} = 34$$

It is not valid.

Thus, the web does not belong to class 1.

Check if the web belongs to class 2.

$$\alpha > 0.5 \quad \Rightarrow \quad \frac{c}{t} = \frac{426}{10.2} = 41.7 \leqslant \frac{456\varepsilon}{13\alpha - 1} = \frac{456 \cdot 0.92}{13 \cdot 0.90 - 1} = 39.2$$

It is not valid.

Thus, the web does not belong to class 2.

Therefore, the web belongs to class 3.

The flange belongs to class 1 (see cases 1 and 2).

Finally, the cross-section belongs to class 3.

9.2.4 Conclusive results

Classification of Web-Flange-Cross-section			
	N	M_y	$N+M_y$
Web	4	1	3
Flange	1	1	1
Cross-section	4	1	3

Remark 2. For the combination of (N, M_y, M_z), the cross-section classification is separately examined for ($N + M_y$) and ($N + M_z$) and the corresponding less favorable class (i.e. the highest class) is considered. This procedure is suggested since the determination of the plastic stress distribution under the combination (N, M_y, M_z) is cumbersome.

9.3 Example: Classification of a box girder cross-section

The welded box girder cross-section shown in Fig. 9.10 is to be classified for the following cases: a) bending about *yy* axis, and b) bending about *zz*. axis. Steel grade S 355.

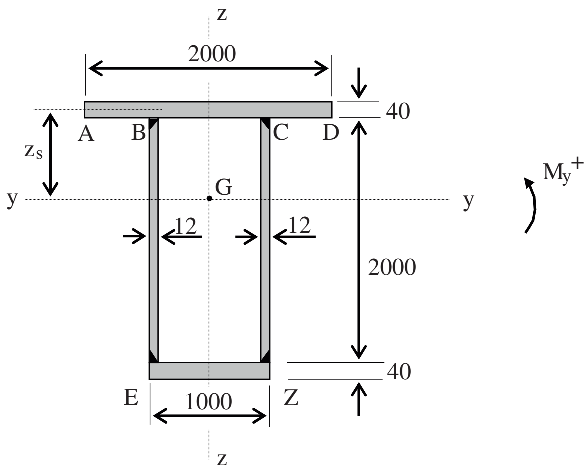


Fig. 9.10. Geometry of the cross-section

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.3.1 Cross-section area and center of gravity of the cross-section

Cross-section area

$$A = 200 \cdot 4 + 200 \cdot 2.4 + 100 \cdot 4 = 1680 \text{ cm}^2$$

Distance between the center of gravity of the cross-section G and the center of gravity of the upper flange

$$z_s = \frac{200 \cdot 2.4 \cdot 102 + 100 \cdot 4 \cdot 204}{1680} = 77.7 \text{ cm}$$

9.3.2 Classification for M_y^+ moments (bending about y-y axis, the upper flange in compression)

$$Steel : S355 \Rightarrow \varepsilon = \sqrt{\frac{235}{355}} = 0.81$$

Flange:

- Element BC (internal)

$$c = 1000 - 2 \cdot 12 = 976 \text{ mm} \qquad t = 40 \text{ mm}$$

$$\frac{c}{t} = \frac{976}{40} = 24.4 < 33\varepsilon = 33 \cdot 0.81 = 26.7$$ Tab. 5.2. sheet 1

Element BC belongs to class 1.

- Parts AB and CD (external)

$$c = \frac{2000 - 1000}{2} = 500\text{ mm}, t = 40\text{ mm}$$

$$\frac{c}{t} = \frac{500}{40} = 12.5 > 14\varepsilon = 11.3$$ Tab. 5.2. sheet 2

Elements AB and CD belong to class 4.
Therefore, flange belongs to class 4.
Web (Elements CZ and BE, see Fig. 9.10):

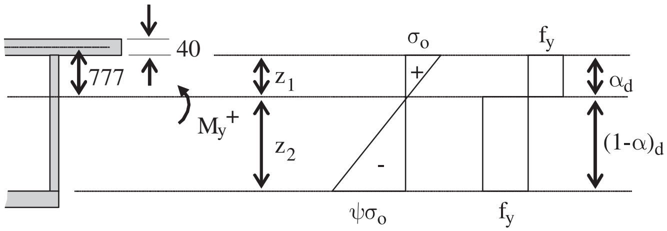


Fig. 9.11. Normal stresses due to M_y^+

a) For an elastic stress distribution

$$z_1 = 777 - \frac{40}{2} = 757\text{ mm}$$
$$z_2 = 2000 - 757 = 1243;\text{mm}$$
$$\psi = -\frac{1243}{757} = -1.64 < -1$$

$$\frac{c}{t} = \frac{2000}{12} = 167 < 62\varepsilon(1-\psi)\sqrt{-\psi} =$$
$$= 62 \cdot 0.81 \cdot (1+1.64) \cdot \sqrt{1.64} = 170$$ Tab. 5.3.1. sheet 1

So, the web is at least class 3.

b) For a plastic stress distribution

It will be examined if the web belongs to a lower class.
Position of plastic neutral axis:

$$200 \cdot 4 + 2 \cdot 1.2\alpha d = 2 \cdot 1.2(d - \alpha d) + 100 \cdot 4$$
$$200 \cdot 4 + 2 \cdot 1.2\alpha 200 = 2 \cdot 1.2 \cdot 200(1-\alpha) + 100 \cdot 4 \Rightarrow \alpha = 0.0833 < 0.5$$

$$\frac{c}{t} = 167 < \frac{36\varepsilon}{\alpha} = \frac{36 \cdot 0.81}{0.0833} = 350$$ Tab. 5.2. sheet 1

So, the web belongs to class 1. and for M_y^+ the whole cross-section belongs to class 4.

9.3.3 Classification for M_y^- moments (bending about y-y axis, the lower flange in compression)

Flange (Element EZ, Fig. 9.10):

$$\frac{c}{t} = \frac{1000-24}{40} = 24.4 < 33\varepsilon = 33 \cdot 0.81 = 26.7$$ Tab. 5.2. sheet 1

So, the flange belongs to class 1.

Web (Elements CZ and BE).

According to paragraph 9.3.2:

$$\psi = -\frac{757}{1243} = -0.609 < -1$$

$$\frac{c}{t} = 167 > \frac{42\varepsilon}{0.67+0.33\psi} = \frac{42 \cdot 0.81}{0.67+0.33(-0.609)} = 72.5$$ Tab. 5.2. sheet 1

So, the web belongs to class 4. and for M_y^- the entire cross-section belongs to class 4.

9.3.4 Classification for M_z moments (bending about z-z axis)

Due to the symmetry about z-z axis, there is no difference between M_z^+ and M_z^- moments.
Flange (Element CZ or BE):

$$\frac{c}{t} = \frac{2000}{12} = 167 > 42\varepsilon = 42 \cdot 0.81 = 34.0$$ Tab. 5.2. sheet 1

The flange belongs to class 4.

Since the maximum stress due to moment M_z appears at point A (or D), the stress at elements CZ or BE, for an elastic stress distribution, is lower than f_y. The stress in these elements is:

$$\sigma_{com \cdot Ed} = \frac{200/2}{100/2} \cdot f_y, \quad \frac{f_y}{\sigma_{com \cdot Ed}} = 2$$

Thus, coefficient ε could be increased by the ratio:

$$\sqrt{\frac{f_y/\gamma_{Mo}}{\sigma_{com \cdot Ed}}} = \sqrt{\frac{2}{1}} = \sqrt{2} \text{ or}$$ 5.5.2(9)

$$\varepsilon = 0.81 \cdot \sqrt{2} = 1.15.$$

Using the new value of ε:

$$\frac{c}{t} = 167 > 42 \cdot 1.15 = 48.3$$

and therefore, the flange remains of class 4.

Web:

- Element BC or EZ

$$\psi = -1$$

$$\frac{c}{t} = \frac{1000-24}{40} = 24.4 < 72\varepsilon = 72 \cdot 0.81 = 58.3$$ Tab. 5.2. sheet 2

Element BC belongs to class 1.

- Element CD

$$\psi = \frac{500}{1000} = 0,5$$

$$k_\sigma = 0.57 - 0.21\psi + 0.07\psi^2 = 0.57 - 0.21 \cdot 0.5 + 0.07 \cdot 0.5^2 = \\ = 0.4825$$ EN 1993-1-5

$$\frac{c}{t} = \frac{500}{40} = 12.5 > 21\varepsilon\sqrt{k_\sigma} = 21 \cdot 0.81 \cdot \sqrt{0.4825} = 11.8$$ Tab. 5.2. sheet 2

Element CD belongs to class 4.

Finally, the web belongs to class 4 and thus for moments M_z the entire cross-section belongs to class 4.

9.3.5 Conclusive results

Moments	M_{y+}	M_{y-}	M_z
Web	1	4	4
Flange	4	1	4
Cross-section	4	4	4

9.4 Example: Bending of a simply supported beam with rolled cross-section

Determine the lightest required IPE cross-section, for the laterally restrained simply supported beam shown in Fig. 9.12. The beam AB belongs to an accessible roof of an office building. The beam is loaded by a permanent load g = 3 kN/m, a uniformly distributed live load p = 5 kN/m and snow load s = 2 kN/m. Steel grade S 235.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

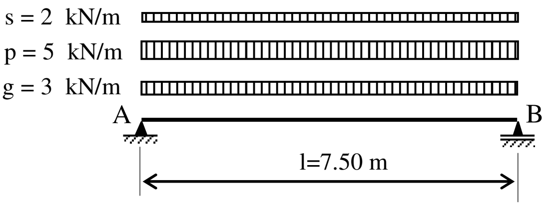


Fig. 9.12. Simply supported beam subjected to permanent and variable loading

9.4.1 Design actions EN 1990

a) The live load as the main variable action

$$q_{Ed} = \gamma_g g + \gamma_q p + \gamma_q \psi_o s = 1.35 \cdot 3 + 1.50 \cdot 5 + 1.50 \cdot 0.50 \cdot 2 = 13.05\,\text{kN/m}$$

b) The snow load as the main variable action

$$q_{Ed} = \gamma_g g + \gamma_q s + \gamma_q \psi_o p = 1.35 \cdot 3 + 1.50 \cdot 2 + 1.50 \cdot 0.70 \cdot 5 = 12.30\,\text{kN/m}$$

Combination a) is more unfavourable.

The partial safety factors γ and the combination factors ψ for the loads should be taken from EN 1990 and the National Annexes.

9.4.2 Cross-section selection based on the bending capacity

Design moment

$$M_{Ed} = \frac{1}{8} \cdot q_{Ed} \cdot l^2 = \frac{1}{8} \cdot 13.05 \cdot 7.50^2 = 91.8\,\text{kNm}$$

Required section modulus

$$M_{c.Rd} \geqslant M_{Ed}$$ Eq. 6.12

$$W_{pl} f_y / \gamma_{Mo} \geqslant M_{Ed}$$ Eq. 6.13

(assuming that the finally chosen cross-section belongs to class 1 or 2)

$$W_{pl} \geqslant 1.00 \cdot 91.8 \cdot 100/23.5 = 390.6\text{cm}^3$$

An IPE 270 cross-section is selected (W_{pl} = 484 cm^3)

Cross-section classification

$$\varepsilon = \sqrt{\frac{235}{235}} = 1.00$$

Web:

$$\frac{c}{t_w} = 219/6.6 = 33.2 < 72\varepsilon = 72$$ Tab. 5.2. sheets 1 and 2

Flange:

$$\frac{c}{t_f} = (67.5 - 3.3 - 15)/10.2 = 4.8 < 9\varepsilon = 9$$

Thus, the cross-section belongs to class 1 and the above provisory assumption regarding the use of plastic section modulus to calculate the bending strength is verified.

9.4.3 Check of shear strength

$$V_{Ed} = \frac{1}{2} \cdot 13.05 \cdot 7.50 = 48.9 \text{ kN}$$

Shear area

$$A_v = 45.9 - 2 \cdot 13.5 \cdot 1.02 + (0.66 + 2 \cdot 1.5)1.02 = 22.09 \text{ cm}^2$$
$$> (27 - 2 \cdot 1.02)0.66 = 16.47 \text{ cm}^2$$ 6.2.6(3)

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{Mo}} = \frac{22.09 \cdot 23.5}{\sqrt{3} \cdot 1.00} = 299.7 \text{ kN} > V_{Ed} = 48.9 \text{ kN}$$ 6.2.6(2)

It is not necessary to verify the web in shear buckling since:

$$h_w/t_w = (27 - 2 \cdot 1.02)/0.66 = 37.8 < 72\varepsilon/\eta = 72$$ Eq. 6.22

9.4.4 Check for serviceability limit state

Since EN 1993-1-1 (see 7.2.1) does not define any numerical values for the limits of vertical deflections, the provisions of the National Annexes or of the project specification, as well as the general principles included in EN 1990-Annex A1.4 should be followed.

The following limits are adopted in this example:

a) Maximum deflection $\delta_{\max}$ equal to $l/250$
b) Deflection δ_2 due to variable actions equal to $l/300$.

The deflections will be calculated for the characteristic combination of actions.

Serviceability limit state load:

$$q_{ser} = g + p + \psi_o s = 3.0 + 5.0 + 0.5 \cdot 2 = 9.0 \text{ kN/m}$$

and the corresponding deflections are:

$$\delta_{\max} = \frac{5}{384} \frac{q_{ser} \cdot l^4}{E \cdot I_y} = \frac{5}{384} \frac{9.0 \cdot 750^4}{21000 \cdot 5790 \cdot 100} = 3.05 \text{ cm} > \frac{750}{250} = 3.00 \text{ cm}$$

$$\delta_2 = \frac{5}{384} \frac{6.0 \cdot 750^4}{21000 \cdot 5790 \cdot 100} = 2.03 \text{ cm} < \frac{750}{300} = 2.50 \text{ cm}$$

Thus, the selected cross-section is not adequate for the serviceability limit state.

9.4.5 Alternative solutions

a) A larger cross-section is selected, i.e. IPE 300.

$$\delta_{\max} = 3.05 \frac{5790}{8360} = 2.11 \text{ cm} < \frac{750}{250} = 3.0 \text{ cm}$$

b) An initial precamber in the unloaded beam IPE 270 is applied.

The precamber in the beam is taken equal to δ_o= 1 cm (approximately equal to the deflection due to permanent actions).

The check of δ_2 remains as previously.

The maximum deflection is:

$$\delta^*_{\max} = \delta_{\max} - \delta_o = 3.15 - 1.00 = 2.15 < 3.0 \text{ cm}.$$

9.5 Example: Bending of a welded plate girder–Influence of shear force

Verify the capacity of the welded plate girder supported as shown in Fig. 9.13a, with a cross-section shown in Fig 9.13b. The design load is equal to q_{Ed} = 110 kN/m and the steel grade is S 235. The beam is laterally restrained.

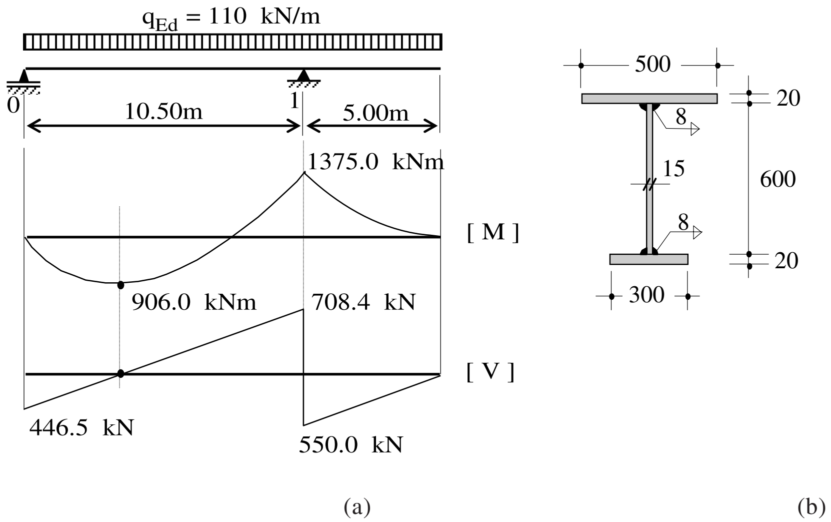


Fig. 9.13. (a) Laterally restrained welded plate girder and bending moments and shear forces diagrams. (b) Welded cross-section of the beam

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.5.1 Internal moments and forces

The distribution of internal bending moments and shear forces due to the design load q_{Ed} is shown in Fig. 9.13a.

9.5.2 Consecutive stages of the cross-section plastification

The consecutive stages of the cross-section plastification are shown in Fig. 9.14 (the upper flange of the span and the lower flange in the area of the support 1 are under compression).

9.5.3 Check of the cross-section at support 1

9.5.3.1 Plastic moment of the cross-section (stress diagram (4) in Fig. 9.14)

Position of the plastic neutral axis (it divides the cross-section in two parts of equal area)

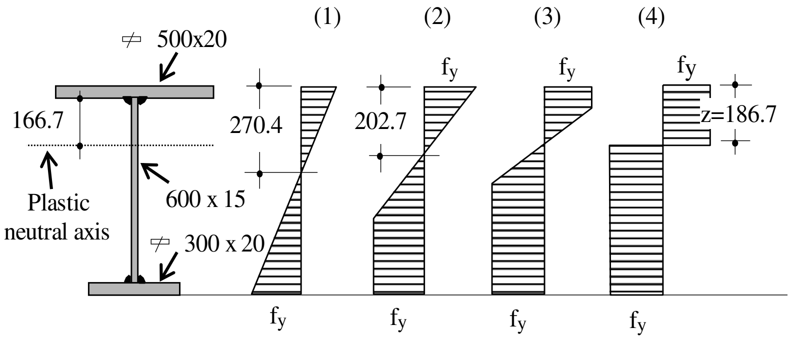


Fig. 9.14. Stress distribution at different loading stages

$$z = 18.67 \text{ cm}$$

Plastic section modulus:

$$W_{pl} = S_1 + S_2 = 50 \cdot 2 \cdot 17.67 + 1.5 \cdot \frac{1}{2} \cdot 16.67^2 + 30 \cdot 2 \cdot (63 - 18.67) + \\ + 1.50 \cdot \frac{1}{2} \cdot (60 - 16.67)^2 = 6043.3 \text{ cm}^3$$

Plastic bending moment

$$M_{pl} = W_{pl} f_y = 6043.3 \cdot 23.5 = 142018 \text{ kNcm} = 1420.2 \text{ kNm}$$

9.5.3.2 Cross-section classification

Lower flange: Tab. 5.2 sheet 2

$$c = \frac{1}{2}(300 - 15) - 8\sqrt{2} = 131.2 \text{ mm}$$

$$\frac{c}{t_f} = 131.2/20 = 6.6 < 9(\varepsilon = 1.0) \qquad \text{Class1}$$

Web (Fig. 9.14. diagram 4): Tab. 5.2 sheet 1

$$\alpha = (60 - 16.67)/60 = 0.722 > 0.50$$

$$\frac{c}{t_w} = \frac{600 - 2 \cdot 8\sqrt{2}}{15} = \frac{577.4}{15} = 38.5 < \frac{396}{13 \cdot 0.722 - 1} = 47.2 \quad \text{Class1}$$

Therefore, the cross-section belongs to class 1 and can develop its plastic moment.

9.5.3.3 Design strength in shear 6.2.6

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{Mo}} = \frac{90 \cdot 23.5}{\sqrt{3} \cdot 1.00} = 1221 kN \qquad \text{Eq. 6.18}$$

where $A_v = 60 \cdot 1.5 = 90\ \mathrm{cm}^2$. 6.2.6(3)

$$V_{Ed} = 708.4\ \mathrm{kN} > 0.50 V_{pl,Rd} = 610.5\ \mathrm{kN}$$ Eq. 6.22

Due to the presence of this shear force, a reduction in the plastic bending moment of the cross-section should be applied. Besides, since

$$\frac{h_w}{t_w} = \frac{600}{15} = 40 < 72\frac{\varepsilon}{\eta} = 72$$

The plastic shear force $V_{pl,Rd}$ can be developed without any presence of shear buckling.

9.5.3.4 Reduction factor of the plastic moment due to shear force

$$\rho = \left(\frac{2 \cdot 708.4}{1221} - 1\right)^2 = 0.0257$$ 6.2.8

9.5.3.5 Verification of the cross-section

The cross-section capacity is calculated using a reduced yield stress for the web:

$$(1-\rho)f_y = (1-0.0257) \cdot 23.5 = 22.9\ \mathrm{kN/cm}^2$$

New position of the plastic neutral axis: since this axis divides the cross-section in two parts of equal area, following equation applies (x is the distance between the plastic neutral axis and the lower fiber of the top flange):

$$50 \cdot 2 \cdot 23.5 + 1.5x \cdot 22.9 = 30 \cdot 2 \cdot 23.5 + 1.5(60-x) \cdot 22.9 \text{ and}$$
$$x = 16.32\ \mathrm{cm}$$

The web of the cross-section belongs, in this case, also to class 1

Accordingly:

$$\alpha = (60 - 16.32)/60 = 0.728 > 0.50$$
$$\varepsilon = (235/229)^{0.5} = 1.013 \text{ and}$$
$$\frac{c}{t_w} = 38.5 < 396 \cdot 1.013/(13 \cdot 0.728 - 1) = 47.4$$

The plastic moment of the cross-section equals to:

$$50 \cdot 2 \cdot 23.5 \cdot 17.32 + 1.5 \cdot 16.32^2 \cdot 22.9 \cdot \frac{1}{2} + 30 \cdot 2 \cdot 23.5 \cdot (61 - 16.32) +$$
$$+ 1.5 \cdot 43.68^2 \cdot 22.9 \cdot \frac{1}{2} = 141044\ \mathrm{kNcm} = 1410.4\ \mathrm{kNm}$$

and $M_{Ed} = 1375.0 < M_{V,Rd} = 1410.4$ kNm.

9.5.4 Check of the cross-section capacity in the span (position of maximum bending moment)

9.5.4.1 Cross-section classification

Upper flange

$$c = \frac{1}{2}(500 - 15) - 8\sqrt{2} = 231.2\text{mm}$$

$$\frac{c}{t_f} = \frac{231.2}{20} = 11.6 \begin{matrix} > 10\varepsilon = 10 \\ < 14\varepsilon = 14 \end{matrix} \qquad \text{Class 3}$$

Tab. 5.2. sheet 2

The web has been classified in the previous case which is worst, as class 1.

Therefore, the cross-section is classified as class 3 and the bending moment corresponds to stress diagram (2) in Fig. 9.14.

9.5.4.2 Calculation of yielding moment

The yielding moment that corresponds to stress diagram (2) is equal to:

$$M_{el,y} = 1350.8 \text{ kNm}$$

9.5.4.3 Capacity of the design strength in bending

Design strength:

$$M_{c,Rd} = M_{el,y}/\gamma_{Mo} = 1350.8/1.0 = 1350.8 \text{ kNm}$$

Eq. 6.14

Check:

$$M_{Ed} = 906.0 < M_{c,Rd} = 1350.8 \text{ kNm}.$$

9.6 Example: Design resistance to bending of a thin walled plate girder

Determine the design resistance to bending of a laterally restrained simply supported plate girder. The cross-section of the plate girder is shown in Fig. 9.15a. The loading is applied at the cross-section's vertical axis of symmetry. Steel grade S 275.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-5, unless otherwise is written.

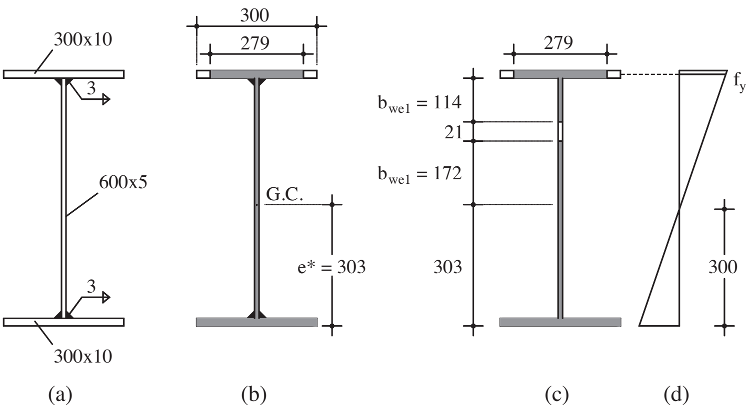


Fig. 9.15. (a) Gross cross-section of the plate girder, (b) cross-section with effective flange, (c) Effective cross-section and (d) stress diagram at the ultimate limit state

9.6.1 Cross-section classification

$$\varepsilon = \sqrt{\frac{235}{275}} = 0.924$$

Web: EN 1993-1-1. Tab. 5.2. sheet 1

$$\frac{c}{t} = (600 - 2 \cdot 3 \cdot \sqrt{2})/5 = 118 > 124\varepsilon = 114.6$$

The web belongs to class 4.

Flange: EN 1993-1-1. Tab. 5.2. sheet 2

$$\frac{c}{t} = (150 - 2.5 - 3 \cdot \sqrt{2})/10 = 14.3 > 14\varepsilon = 12.9$$

The flange belongs to class 4. and the entire section belongs to class 4.

9.6.2 Effective width of the flange

The upper flange is subjected to uniform compressive stress, so $\psi = +1$ and the plate buckling coefficient is $k_\sigma = 0.43$. Tab. 4.2

Relative slenderness of the flange

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} = \frac{143/10}{28.4 \cdot 0.924 \cdot \sqrt{0.43}} = 0.831 \qquad 4.4(2)$$

Reduction factor

$$\rho = \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} = \frac{0.831 - 0.188}{0.831^2} = 0.931 < 1.0 \qquad \text{Eq.4.3}$$

$$b_{eff} = \rho \cdot c = 0.931 \cdot 143 = 133 \text{ mm}$$

and total effective width of the flange

$$b_{f,eff} = 2 \cdot 133 + 5 + 2 \cdot 3 \cdot \sqrt{2} = 279 \text{ mm}$$

9.6.3 Stress distribution and effective area of the web

The determination of the effective area in the compressed parts of the web should be obtained considering the reduced width of the compressed flange and the gross area of the web (see Fig. 9.15b). 4.4(3)

The center of gravity of the cross-section shown in Fig. 9.15b is derived from following equation:

$$27.9 \cdot 1 \cdot 61.5 + 60 \cdot 0.5 \cdot 31 + 30 \cdot 0.5 = (30 + 27.9 + 60 \cdot 0.5)e^*$$

and

$$e^* = 30.3 \text{ cm (see Fig. 6.1b)},$$
$$\psi = -293/307 = -0.954$$

$$k_\sigma = 7.81 - 6.29\psi + 9.78\psi^2 = 22.71 \text{ (plate buckling coefficient)}$$ Tab.4.1

$$\bar{\lambda}_p = \frac{(60 - 2 \cdot 0.3 \cdot \sqrt{2})/0.5}{28.4 \cdot 0.924 \cdot \sqrt{22.71}} = 0.946$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3 + \psi)}{\bar{\lambda}_p^2} = \frac{0.946 - 0.055(3 - 0.954)}{0.946^2} = 0.93 < 1.0$$ Eq. 4.2

$$b_{eff} = b_c \cdot \rho = 0.93 \cdot 307 = 286 \text{ mm}$$ Tab. 4.1

therefore:

$$b_{w,e1} = 0.4 \cdot 286 = 114 \text{ mm}$$
$$b_{w,e2} = 0.6 \cdot 286 = 172 \text{ mm}$$

Non effective length of the web: $307 - 286 = 21$ mm

The effective cross-section is presented in Fig. 9.15c.

9.6.4 Geometrical properties of the effective cross-section

Position of the center of gravity:

$$27.9 \cdot 61.5 + 11.4 \cdot 0.5 \cdot 55.3 + (60 - 11.4 - 2.1) \cdot 0.5 \cdot 24.25 +$$
$$+ 30 \cdot 1 \cdot 0.5 = (27.9 + 11.4 \cdot 0.5 + 46.5 \cdot 0.5 + 30) \cdot e,$$

and $e = 300$ mm (see Fig. 9.15c).

Second moment of area about the centroidal axis (Fig. 9.15c):

$$I_{y,eff} = 27.9 \cdot (61.5 - 30.0)^2 + 11.4 \cdot 0.5 \cdot (55.3 - 30.0)^2 + \frac{1}{12} \cdot 0.5 \cdot 11.4^3 + \\ + \frac{1}{12} \cdot 0.5 \cdot 46.2^3 + 46.2 \cdot 0.5(24.1 - 30.0)^2 + 30 \cdot 1 \cdot 29.0^2 = \\ = 61537 \text{ cm}^4$$

The stress distribution considered corresponds to the attainment of yield stress in the mid plane of the compression flange (see Fig. 9.15d). 4.2(2)

Effective section modulus:

$$W_{eff} = I_{y,eff}/y_{c,\max} = 61537/(62 - 30.0) = 1923 \text{ cm}^3$$

9.6.5 Design resistance for bending

$$M_{c,Rd} = W_{eff} f_y/\gamma_{M0} = 1923 \cdot 27.5/1.00 = \\ = 52882 \text{ kNcm} = 528.8 \text{ kNm},$$

EN 1993-1-1. 6.2.5(2)

Remark 3. The influence of shear lag can be neglected, assuming that the following limit is valid:

$$b_o = \frac{30}{2} = 15 \text{ cm} < \frac{L_e}{50}$$

3.1(1)

9.7 Example: Design of a beam with alternative methods

Verify the capacity of the continuous beam shown in Fig. 9.16. The loading of the beam consists of a permanent load $g = 0.13$kN/m and a uniformly distributed snow load $s = 2.65$ kN/m. The beam is laterally restrained. The cross-section is an IPE 120 and the steel grade is S 235.

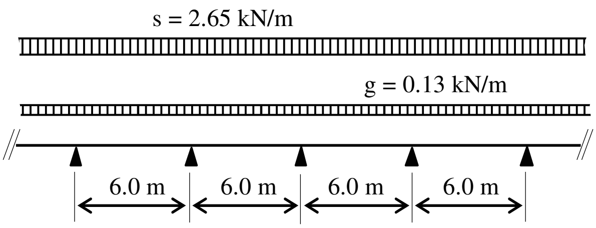


Fig. 9.16. Continuous beam with equal spans

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.7.1 Elastic analysis

5.4.2

Design load:

$$q_{Ed} = 1.35 \cdot g + 1.50 \cdot s = 1.35 \cdot 0.13 + 1.5 \cdot 2.65 = 4.15 kN/m.$$ EN 1990

Design moments

$$\text{Support:} \quad M_{Ed} = \frac{q_{Ed} l^2}{12} = \frac{4.15 \cdot 6^2}{12} = 12.45 \text{ kNm}$$

$$\text{Span:} \quad M_{Ed} = \frac{q_{Ed} l^2}{24} = \frac{4.15 \cdot 6^2}{24} = 6.23 \text{ kNm}$$

Cross-section classification Tab. 5.2

$$\text{Steel S 235.} \quad \varepsilon = \sqrt{\frac{235}{235}} = 1$$

Flange: $t_f = 6.3$ mm

$$c = \frac{64 - 4.4}{2} - 7 = 22.8 \text{ mm}$$

$$\frac{c}{t_f} = \frac{22.8}{6.3} = 3.6 < 9\varepsilon = 9 \cdot 1 = 9$$

The flange belongs to class 1

Web: c = 93 mm, t_w = 4.4 mm

$$\frac{c}{t_w} = \frac{93}{4.4} = 21.1 < 72\varepsilon = 72 \cdot 1 = 72$$

The web belongs also to class 1 and therefore, the whole section belongs to class 1. 5.5.2(6)

The design resistance for bending could be taken as the plastic moment.

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{60.8 \cdot 23.5}{1.0 \cdot 100} = 14.3 \text{ kNm}$$ Eq. 6.13

Check:

$$M_{Ed} = 12.45 \text{ kNm} < M_{c,Rd} = 14.3 \text{ kNm}$$ Eq. 6.12

Remark 4. If the design resistance for bending was taken as the elastic moment, which is permitted for class 1 and 2 sections, it would be:

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el} f_y}{\gamma_{Mo}} = \frac{53 \cdot 23.5}{1.0 \cdot 100} = 12.45 \text{ kNm}$$ Eq. 6.14

and thus

$$M_{Ed} = 12.45 \text{ kNm} = M_{c,Rd} = 12.45 \text{ kNm}$$

i.e. the section IPE 120 has a limited capacity.

9.7.2 Elastic analysis with redistribution of moments

Since the cross-section of the beam is of class 1. a redistribution of moments up to 15 % is permitted. 5.4.1(4)

Design moments

15 % of maximum moment at support M_{sup} is redistributed to the span.

Bending moment for redistribution:

$$\Delta M = 15\% M_{sup} = 15\% \cdot 12.45 = 1.87\,\text{kNm}$$
$$\text{Support:}\quad M_{Ed} = 12.45 - 1.87 = 10.58\,\text{kNm}$$
$$\text{Span:}\quad M_{Ed} = 6.23 + 1.87 = 8.10\,\text{kNm}$$

The maximum design moment M_{Ed} = 10.58 kNm is less than $M_{pl,Rd}$ = 14.3 kNm and $M_{el,Rd}$ = 12.45 kNm of section IPE 120.

The check in the last case could be alternatively done through the stresses:

$$\sigma_{X,\,Ed} = \frac{M_{Ed}}{W_{el}} = \frac{1058}{53} = 20.0\,\text{kN/cm}^2 < \frac{f_y}{\gamma_{M0}} = \frac{23.5}{1.0} = 23.5 kN/cm^2$$ Eq. 6.1

9.7.3 Plastic analysis

5.4.3

Check for applicability of plastic analysis

- Cross-section symmetrical about the axis of loading. 5.4.1(2)
- Class 1 cross-section. 5.5.2(1)
- The compression flange is laterally restrained at the areas of plastic hinges. 5.4.3(3)

Therefore, plastic analysis is permitted. 5.4.3

Plastic analysis will be performed based on the 1st order theory of plastic hinges (stereoplastic analysis). 5.4.3(1)

At the limit state, plastic hinges develop simultaneously at span and supports, and the ultimate load q_u is:

$$\frac{q_u l^2}{16} = M_{pl,Rd} \Rightarrow \frac{q_u 6^2}{16} = 14.3\,\text{kNm} \Rightarrow$$
$$q_{u,Rd} = \frac{14.3 \cdot 16}{6^2} = 6.36\,\text{kN/m}$$

Check: $q_{u,Rd} = 6.36$ kN/m $> q_{Ed} = 4.15$ kN/m, and the cross-section is sufficient.

9.7.4 Verification of shear force

Design shear force (the same in all methods of analysis)

$$V_{Ed} = \frac{q_{Ed} l}{2} = \frac{4.15 \cdot 6}{2} = 12.45\,\text{kN}$$

Shear area of the web

$$A_V = 13.2 - 2 \cdot 6.4 \cdot 0.63 + (0.44 + 2 \cdot 0.7) \cdot 0.63 = 6.30 \text{ cm}^2$$ 6.2.6(2)

and $A_V > nh_w \quad t_w = 1 \cdot 9.3 \cdot 0.44 = 4.09 \text{ cm}^2$

Plastic design shear resistance

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{Mo}} = \frac{6.30 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 85.5 \text{ kN}$$ Eq. 6.18

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{12.45}{85.5} = 0.14 < 0.50$$ Eq. 6.17

Thus, no reduction of the design moments due to shear force is required.

In case of plastic analysis, the corresponding load on the beam could be up to q_{Ed} = 6.36 kN/m, the corresponding design shear force would be:

$$V_{Ed} = 12.45 \cdot \frac{6.36}{4.15} = 19.1 \text{ kN and}$$ 5.6(2b)

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{19.1}{85.5} = 0.22.$$

Since this value is greater than 0.10 web stiffeners sould be provided at supports.

Remark 5. Shear buckling verification is not required, since: 5.4.6

$$\frac{hw}{t_w} = \frac{10.74}{0.44} = 24.4 < 72\frac{\varepsilon}{\eta} = 72 \cdot 1 = 72$$ Eq. 6.22

in which $h_w = 12 - 2 \cdot 0.63 = 10.74$ cm.

9.8 Example: Cross-section under simultaneous bending, shear force and axial force

Verify the adequacy of the simply supported laterally restrained beam shown in Fig. 9.17. with a HEB 400 cross-section. The beam is loaded by vertical design loads q_{Ed} = 7 kN/m and P_{Ed} = 900 kN, and a design axial force N_{Ed} = 850 kN. Steel grade S 235.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.8.1 Determination of the internal forces and moments

The verification will be performed at midspan (worst position). See also remark 1 at the end of the example.

Internal moments and forces:

$$M_{Ed} = \frac{1}{8} \cdot 7 \cdot 2.5^2 + \frac{1}{4} \cdot 900 \cdot 2.5 = 568 \text{ kNm}$$

$$V_{Ed} = \frac{1}{2} \cdot 900 = 450 \text{ kN}$$

$$N_{Ed} = 850 \text{ kN}$$

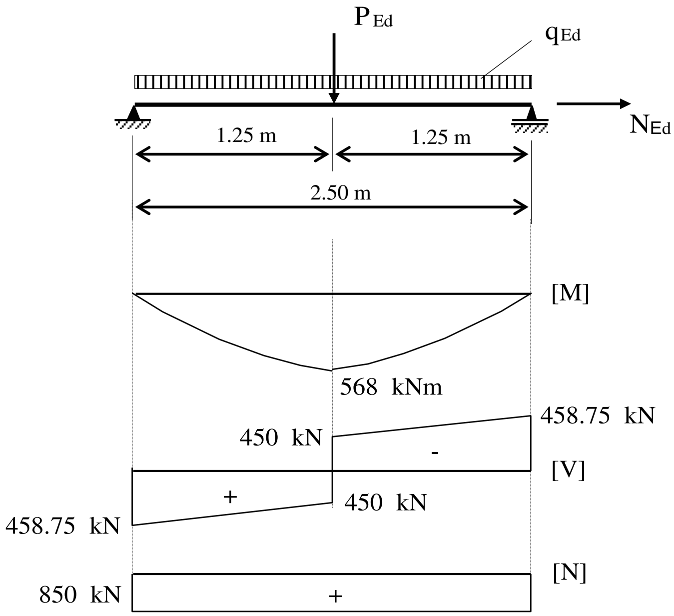


Fig. 9.17. Simply supported beam under transversal and axial forces. Diagrams of internal moments and forces

9.8.2 Reduction factor ρ due to the presence of shear force

Shear area 6.2.6(3)

The shear area is assumed to be a rectangle with dimensions the height of the section and the thickness of the web.

$$A_v = 40 \cdot 1.35 = 54.0\,\text{cm}^2$$ 6.2.6(2)

Design plastic shear resistance

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{M0}} = \frac{54.0 \cdot 23.5}{\sqrt{3} \cdot 1.00} = 732.6\,\text{kN}$$

Since

$$V_{Ed} = 450kN < V_{pl,Rd}, \text{ but } \quad V_{Ed} > \frac{1}{2} \quad V_{pl,Rd} = 366.3\ kN,$$

a reduction of the cross-section moment resistance due to shear force should be considered.

The reduction factor is:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2 \cdot 450}{732.6} - 1\right)^2 = 0.052$$ 6.2.8(3)

The reduced yield strength for the shear area is:

$$f_{yw} = (1-\rho) f_y = (1-0.052) \cdot 23.5 = 22.2\,\text{kN/cm}^2$$ 6.2.10

9.8.3 Reduced design resistance moment due to shear force

The cross-section belongs to class 1 (For the classification procedure see examples 9.2 and 9.3).

The reduced design resistance moment should be calculated using the reduced stress f_{yw} for the web and f_y for the flanges.

Therefore:

$$M_{v,Rd} = [2 \cdot (30 - 1.35) \cdot 2.4 \cdot 18.8 \cdot 23.5 + 2 \cdot 20 \cdot 1.35 \cdot 10 \cdot 22.2]/1.00 = \\ = 72744 \text{ kNcm} = 727.4 \text{ kNm}$$

In the above formula, the contribution of the adjustment areas between flange and web has been omitted for simplicity reasons (safe side approach). This moment $M_{V,Rd}$ is comparable to the full plastic design moment:

$$M_{pl,Rd} = W_{pl} f_y / \gamma_{Mo} = 3232 \cdot 23.5/100 \cdot 1.00 = 759.5 \text{ kNm}$$

Reduced design plastic resistance to tension force:

$$N_{pl,Rd} = [54.0 \cdot 22.2 + (198 - 54.0) \cdot 23.5]/1.00 = 4583 \text{ kN}$$

comparable to the non-reduced value:

$$N^*_{pl,Rd} = 198 \cdot 23.5/1.00 = 4653 \text{ kN}.$$

9.8.4 Reduced design resistance moment due to tension force

$$M_{Ny,Rd} = M_{pl,y,Rd}(1 - n)/(1 - 0.5\alpha) \leqslant M_{pl,y,Rd}$$ Eq.6.36

$$n = N_{Ed}/N_{pl,Rd} = 850/4583 = 0.185$$
$$\alpha = (A - 2bt_f)/A = (198 - 2 \cdot 30 \cdot 2.4)/198 = 0.273 < 0.50$$

and therefore: 6.2.9.1(5)

$$M_{Ny,Rd} = 727.4(1 - 0.185)/(1 - 0.5 \cdot 0.273) = 727.4 \cdot 0.815/0.864 = 686.1 \text{ kNm}$$

9.8.5 Verification of the cross-section

$$M_{Ed} = 568 \text{ kNm} < M_{N.y,Rd} = 686.1 \text{ kNm}$$

Remark 6. The above described procedure should be performed at all cross-sections and not only at the position of maximum bending moment.

Remark 7. In the case of a cross-section (and not of a member) verification, the procedure for axial compression force is the same as above.

Remark 8. It is assumed that the web of the beam is adequate against any kind of local failure (for example through stiffeners, if needed) at the position of the concentrated load.

9.9 Example: Beam under biaxial bending and axial tension force

Verify the capacity of the HEA 140 cross-section of a simply supported beam shown in Fig. 9.18. The beam is laterally restrained at supports and at each 1/3 of the span. Steel quality S 235.

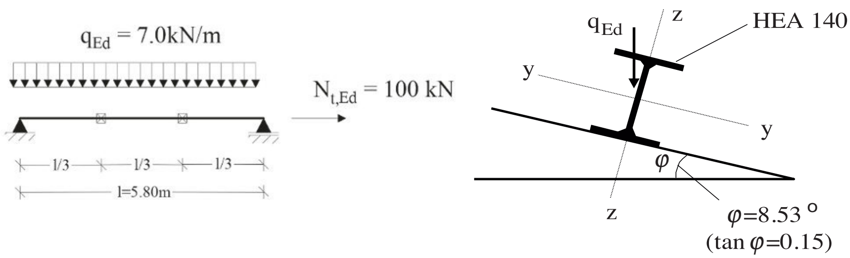


Fig. 9.18. Simply supported beam under biaxial bending and tension

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.9.1 Influence of the axial load on the plastic moments

6.2.9

Plastic axial load of the cross-section:

$$N_{pl,Rd} = Af_y/\gamma_{M0} = 31.4 \cdot 23.5/1.00 = 737.9 \text{ kN}$$

Since

$$N_{Ed} = 100 \text{ kN} > \frac{0.5\, h_w t_w f_y}{\gamma_{M0}} = $$

$$= \frac{0.5\,(13.3 - 2 \cdot 0.85) \cdot 0.55 \cdot 23.5}{1.00} = 75.0 \text{ kN}$$

Eq. 6.34

a reduction due to the axial load on the plastic moment about y-y axis should be considered. 6.2.9.1(4)

Besides:

$$N_{Ed} = 100 \text{ kN} < \frac{h_w t_w f_y}{\gamma_{M0}} = 150 \text{ kN}$$

Eq. 6.35

and therefore, no reduction is required for z-z axis.

9.9.2 Reduced plastic moments

6.2.9.1(5)

$$\alpha = (A - 2bt_f)/A = (31.4 - 2 \cdot 14 \cdot 0.85)/31.4 = 0.242 < 0.5 \quad \text{and}$$
$$\eta = N_{Ed}/N_{pl,Rd} = 100/737.9 = 0.136$$

Thus:

$$\begin{aligned} M_{Ny,Rd} &= M_{pl,y,Rd}(1-\eta)/(1-0.5\alpha) = \\ &= \frac{173 \cdot 23.5}{1.00}(1-0.136)/(1-0.5\cdot 0.242) = \\ &= 3996 \text{ kNcm} = 39.96 \text{ kNm} \end{aligned}$$

Eq. 6.36

and $M_{Nz,Rd} = M_{pl,z,Rd} = \frac{83.4 \cdot 23.5}{1.00 \cdot 100} = 19.60$ kNm.

9.9.3 Design bending moments

Design loads

$$\begin{aligned} q_{Edy} &= q_{Ed}\cos 8.53^\circ = 7.0 \cdot 0.989 = 6.92 \text{ kN/m} \\ q_{Ed \cdot z} &= q_{Ed}\sin 8.53^\circ = 7.0 \cdot 0.148 = 1.04 \text{ kN/m} \end{aligned}$$

Design moments

a) Midspan

$$M_{y,Ed} = \frac{1}{8} \cdot 6.92 \cdot 5.80^2 = 29.10 \text{ kNm}$$

$$M_{z,\mathrm{Ed}} = 0.025 \cdot 1.04 \cdot \left(\frac{5.80}{3}\right)^2 = 0.10 kNm = 10 \text{ kNcm}$$

(in lateral direction, the beam is assumed as continuous in three spans).

b) Intermediate position of a lateral restraint

$$M_{y,Ed} = 25.92 \text{ kNm}$$

$$M_{z,\mathrm{Ed}} = 0.100 \cdot 1.04 \cdot \left(\frac{5.80}{3}\right)^2 = 0.39 \text{ kNm}$$

9.9.4 Verification of cross-section capacity

6.2.9.1(6)

The following formula is used:

$$\left[\frac{M_{y,Ed}}{M_{Ny,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{Nz,Rd}}\right]^{\beta} \leqslant 1$$

Eq. 6.41

For I- and H- sections:

$$\alpha = 2. \quad \beta = 5\eta = 5 \cdot 0.136 = 0.68 < 1. \quad \text{and thus } \beta = 1.$$

The interaction formula is written:

a) Midspan

$$\left(\frac{29.10}{39.96}\right)^2 + \frac{0.10}{19.60} = 0.728^2 + 0.005 = 0.54 < 1$$

b) Intermediate position of lateral restrain

$$\left(\frac{25.92}{39.96}\right)^2 + \frac{0.39}{19.60} = 0.649^2 + 0.020 = 0.44 < 1$$

It may be seen than in both positions the uniaxial bending and tension check is more severe than the biaxial one. Accordingly, the utilization level is 0.728 (not 0.54) at midspan and 0.649 (not 0.44) at the position of the lateral restrain.

9.9.5 Verification of lateral-torsional buckling

The elastic critical moment for lateral-torsional buckling is given from the relation:

$$M_{cr} = C_1 \quad \frac{\pi^2 \mathrm{EI}_z}{(kl)^2}\left[\frac{I_w}{I_z} + \frac{(kl)^2 GI_t}{\pi^2 \mathrm{EI}_z}\right]^{0,5}$$ (Literature)

In the case of a uniformly distributed load with two intermediate lateral supports:

$C_1 = 1.365,$ $k = 1/3,$ $l = 580\ \mathrm{cm},$

$E/G = 2.6,$ $E = 21000\ \mathrm{kN/cm^2},$ $I_z = 389\ \mathrm{cm^4},$

$I_w = 15300\ \mathrm{cm^6},$ $I_t = 8.16\ \mathrm{cm^4}$

and therefore: $M_{cr} = 24613\ \mathrm{kNcm}$

(the favorable influence of the axial tension force has been omitted).

Since the relative slenderness for lateral torsional buckling is:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl} \cdot f_y}{M_{cr}}} = \sqrt{\frac{173 \cdot 23.5}{24613}} = 0.40$$ 6.3.2.2

the lateral-torsional buckling effects for hot rolled sections could be neglected. 6.3.2.3

9.10 Example: Lateral-torsional buckling of a plate girder with doubly symmetrical cross-section

A simply supported plate girder AB shown in Fig. 9.19 has simple torsional (forked) restraints at the end supports A and B, without intermediate lateral restraints. Verify the capacity of the plate girder for a uniformly distributed design load q_{Ed} = 80 kN/m applied along the z-z axis, at the center of gravity of the cross-section. Steel grade S 355.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

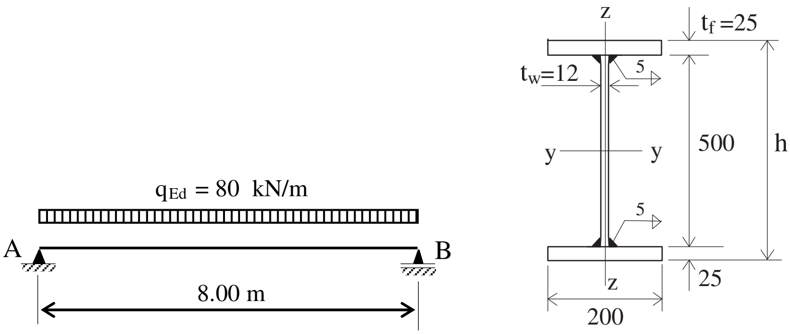


Fig. 9.19. Simply supported plate girder without intermediate lateral supports

9.10.1 Cross-section classification

Tab. 5.2

$$\varepsilon = \sqrt{\frac{235}{355}} = 0{,}814$$

Flange:

$$\frac{c}{t} = (100 - 6 - 5\sqrt{2})/25 = 86.9/25 = 3.5 < 9\varepsilon = 7.3$$

so, the flange belongs to class 1.

Web:

$$\frac{c}{t} = (500 - 2 \cdot 5 \cdot \sqrt{2})/12 = 486/12 = 40.5 < 72\varepsilon = 58.6$$

The web belongs to class 1 and thus the entire section to class 1.

9.10.2 Cross-section verification to bending moment

Plastic section modulus

$$W_{pl} = 2 \cdot (20 \cdot 2.5 \cdot 26.25 + 25 \cdot 1.2 \cdot 12.5) = 3375 \text{ cm}^3$$

Design resistance for bending 6.2.5

$$M_{c,Rd} = 3375 \cdot 35.5/1.00 = 119813 \text{ kNcm}$$ Eq. 6.13

Bending moment verification:

$$M_{Ed} = \frac{1}{8} \cdot 80 \cdot 8.00^2 \cdot 100 = 64000 \text{ kNcm} < M_{c,Rd}$$

9.10.3 Verification to shear force

Design resistance to shear: 6.2.6

$$A_v = 50 \cdot 1.2 = 60 \text{ cm}^2$$

$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{M0} = 60(35.5/\sqrt{3})/1.00 = 1230 \text{ kN}$$ Eq. 6.18

Design shear force:

$$V_{Ed} = \frac{1}{2} \cdot 80 \cdot 8.00 = 320 \text{ kN} \leq V_{pl,Rd} = 1230 \text{ kN}$$ Eq. 6.17

9.10.4 Verification to bending and shear

Reduction of the plastic moment due to shear is not examined, since the shear force is zero at the position of maximum bending moment. No reduction of the plastic moment would be necessary even in the case of coexistence of bending moment and shear force at the same section, since:

$$V_{Ed} = 320kN < 0.50V_{pl,Rd} = 0.50 \cdot 1230 = 615 \text{ kN}$$ 6.2.8(4)

9.10.5 Lateral torsional buckling verification

Eq. 6.3.2

The verification should be done using the relation (6.54) of EN 1993-1-1 and the relative slenderness $\overline{\lambda}_{LT}$. The elastic critical moment for lateral-torsional buckling is calculated through the formula:

$$M_{cr} = C_1 \frac{\pi^2 \mathrm{EI}_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G J_t}{\pi^2 \mathrm{EI}_z} + (C_2\, z_g)^2 \right]^{0,5} - C_2\, z_g \right\}$$ (Literature)

which applies for doubly symmetric cross-sections. The symbols are explained below, while L is the laterally unsupported length (in this example L= 8.00 m).

Second moment of area about z-z axis:

$$I_z = 2 \cdot \frac{1}{12} \cdot 2.5 \cdot 20^3 = 3333 \text{cm}^4$$

Warping constant:

$$I_w = \frac{1}{4} I_z (h - t_f)^2 = \frac{1}{4} \cdot 3333 \cdot 52.5^2 = 2296645 \text{ cm}^6$$ (Literature)

Torsion constant:

$$I_t = \frac{1}{3} \sum b_i t_i^3 = \frac{1}{3} \cdot (2 \cdot 20 \cdot 2.5^3 + 50 \cdot 1.2^3) = 237.1 \text{ cm}^4$$ (Literature)

Modulus of elasticity and shear modulus:

$$E = 21000\,\text{kN/cm}^2$$

$$G = E/2(1+\nu) = 21000/2(1+0.3) = 8077\,\text{kN/cm}^2 \qquad 3.2.6$$

Regarding to supports' conditions, the following coefficients are used: $k = 1.0$ (simple lateral restraints at supports),

$$k_w = 1.0 \text{ (the cross-sections are free of warping at the supports).}$$

Besides: $z_g = z_\alpha - z_s$ in which z_α, z_s the coordinations of the application point of the external loading and of the shear center correspondingly.

The loading is applied at the center of gravity, thus: $z_\alpha = 0$.

Besides, since the center of gravity and the shear center coincide: $z_s = 0$ and therefore: $z_g = z_\alpha - z_s = 0$.

The constants $C_{1.}$ C_2 depend on the loading conditions and may be taken from the literature. In this case: $C_1 = 1.132$. $C_2 = 0.459$.

Elastic critical moment for lateral-torsional buckling:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\frac{k}{k_w}\right]^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} \right\}^{0.5} =$$

$$= 1.132 \frac{\pi^2 \cdot 21000 \cdot 3333}{800^2} \left\{ \frac{2296645}{3333} + \frac{800^2 \cdot 8077 \cdot 237.1}{\pi^2 \cdot 21000 \cdot 3333} \right\}^{0.5}$$

$$= 60643\,\text{kNcm}$$

Relative slenderness:

$$\overline{\lambda}_{LT} = \left[W_{pl} f_y / M_{cr}\right]^{0.5} = (3375 \cdot 35.5/60643)^{0.5} = 1.406 \qquad 6.3.2.2(1)$$

Since: h/b = 550/200 = 2.75 $>$ 2 buckling curve (d) should be used, in which the corresponding imperfection factor is $\alpha_{LT} = 0.76$.

$$\Phi_{LT} = 0.5[1 + 0.76(1.406 - 0.4) + 0.75 \cdot 1.406^2] = 1.623 \qquad \text{Eq. 6.57}$$

$$\chi_{LT} = 1/[(1.623 + (1.623^2 - 0.75 \cdot 1.406^2)^{0.5}] = 0.370$$

Design lateral-torsional buckling resistance moment

$$M_{b \cdot Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} = 0.370 \cdot 3375 \cdot \frac{35.5}{1.0} = 44330\,\text{kNcm} \qquad \text{Eq. 6.55}$$

and

$$M_{\text{Ed}} = 64000 > M_{b,Rd} = 44330\,\text{kNcm}. \qquad \text{Eq. 6.54}$$

The beam in this case is not adequate for lateral-torsional buckling.

Remark 9. The verification at the serviceability limit state and the capacity of the web to shear buckling have been omitted.

Remark 10. For comparison, the following two cases for the point of application of the design load are examined: a) at the middle of the upper flange, b) at the middle of the lower flange of the beam.

a) The design load is applied at the middle of the upper, compression flange
   The load is applied at: z_α = 27.5 cm (the distance z_α is introduced as positive provided that the upper flange is under compression):

$$z_g = z_\alpha = 27.5 \text{ cm}$$

$$M_{cr} = 1.132 \cdot \frac{\pi^2 \cdot 21000 \cdot 3333}{800^2} \cdot \left\{ \left[\frac{2296645}{3333} + \frac{800^2 \cdot 8077 \cdot 237.1}{\pi^2 \cdot 21000 \cdot 3333} + \right.\right.$$
$$\left.\left. + (0.459 \cdot 27.5)^2 \right]^{0.5} - 0.459 \cdot 27.5 \right\} = 47150 \text{ kNcm}$$

$$\bar{\lambda}_{LT} = 1.594, \quad \Phi_{LT} = 1.906. \quad \chi_{LT} = 0.311 \quad \text{and}$$
$$M_{b,Rd} = 37261 \text{ kNcm}$$

   The resistance in lateral-torsional buckling has been reduced approximately 16 % in relation to the initial case.

b) The design load is applied at the middle of the lower, tension flange

$$z_\alpha = -27.5 \text{ cm}, \quad z_g = -27.5 \text{ cm},$$
$$M_{cr} = 77991 \text{ kNcm}, \quad \bar{\lambda}_{LT} = 1.239, \quad \Phi_{LT} = 1.395, \quad \chi_{LT} = 0.437$$
$$M_{b,Rd} = 52358 \text{ kNcm}$$

   The resistance to lateral-torsional buckling has been increased approximately 18 % in relation to the initial case.

9.11 Example: Lateral-torsional buckling of a plate girder with a simply symmetric cross-section

A simply supported plate girder with a 24 m span and simply symmetric I-section shown in Fig. 9.20 is arranged at the supports with simple torsional (forked) restraints without intermediate lateral protection. Verify the capacity of the plate girder for its self-weight. Steel grade S 355.
Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.11.1 Cross-section classification

The plastic neutral axis divides the section in two parts of equal area ($\alpha - \alpha$ axis, Fig. 9.20).
Thus:

$$50 \cdot 3.5 + 2x = 40 \cdot 3.5 + (100 - x) \cdot 2 \text{ and } x = 41.25 \text{ cm}$$

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$$

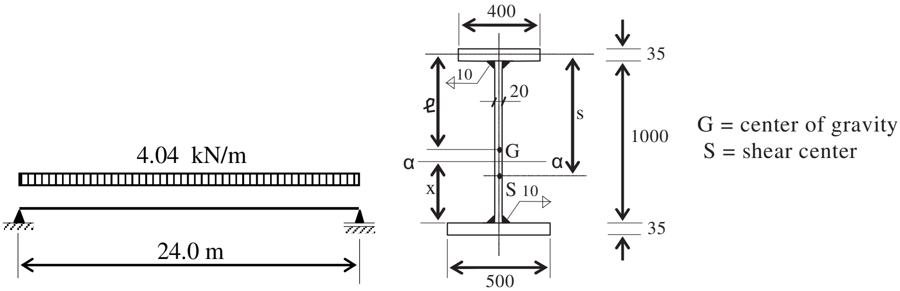


Fig. 9.20. Simply supported plate girder with simply symmetric I-cross-section

Web: Tab. 5.2. sheet 1

$$\alpha = 58.75/100 = 0.5875 > 0.5$$
$$d/t_w = 100/2 = 50 < 456 \cdot 0.81/(13 \cdot 0.5875 - 1) = 55.6$$

and

$$d/t_w = 50 > 396 \cdot 0.81/(13 \cdot 0.5875 - 1) = 48.3$$

The web belongs to class 2.
Flange: Tab. 5.2. sheet 2

$$c = \frac{1}{2} \cdot (400 - 20) - 10 \cdot \sqrt{2} = 175.9$$
$$c/t_f = 175.9/35 = 5.0 < 9 \cdot 0.81 = 7.3$$

The flange belongs to class 1 and the whole section to class 2, so it could develop its plastic moment.

9.11.2 Design moment

Self-weight of the beam:

$$g = (40 \cdot 3.5 + 100 \cdot 2 + 50 \cdot 3.5) \cdot 7.85 \cdot 10^{-3} = 4.04 \text{ kN/m and}$$
$$M_{Ed} = 1.35 \cdot \frac{1}{8} \cdot 4.04 \cdot 24.0^2 = 392.7 \text{ kNm}$$

9.11.3 Elastic critical moment for lateral-torsional buckling

a) center of gravity G of the section
(distance e from the middle axis of the upper flange, Fig. 9.20
Cross-section area

$$A = (40 + 50) \cdot 3.5 + 100 \cdot 2 = 515 \text{ cm}^2$$

The following equation applies:

$$50 \cdot 3.5 \cdot 103.5 + 100 \cdot 2 \cdot 51.75 = 515 \cdot e, \quad \text{and} \quad e = 55.27 \text{ cm}$$

b) Shear center

Second moment of area of the lower flange about minor axis

$$I_{fzu} = \frac{1}{12} \cdot 3.5 \cdot 50^3 = 36458 \text{ cm}^4$$

Second moment of area of the upper flange about minor axis

$$I_{fzo} = \frac{1}{12} \cdot 3.5 \cdot 40^3 = 18667 \text{ cm}^4$$

Second moment of area of the cross-section about minor axis

$$I_z = I_{fzu} + I_{fzo} = 55125 \text{ cm}^4$$

Distance between centroid axes of flanges

$$h = 103.5 \text{ cm}$$

Position of the shear center

(closer to the lower flange, starting from the center of gravity as origin)

$$z_s = \frac{(h-e)I_{fzu} - eI_{fzo}}{I_z} = \frac{(103.5 - 55.27) \cdot 36458 - 55.27 \cdot 18667}{55125} = \text{(Literature)}$$
$$= 13.18 \text{ cm}$$

c) Determination of z_j coefficient

It is generally:

$$z_j = z_\alpha - \frac{\int_A z(y^2 - z^2)dA}{2I_y}$$

where z_a is the coordinate of the point of application of the external loads, and I_y is the second moment of area of the cross-section about its strong axis. In this example, the external load is applied in the center of gravity (i.e. $z_a = 0$) and thus:

$$z_j = z_s - \frac{1}{2}r,$$

$$rI_y = z_s I_z - A_{fo}e^2 + A_{fu}(h-e)^2 + \frac{t_w}{4}[(h-e)^4 - e^4] =$$
$$= 13.18 \cdot 55125 - 140 \cdot 55.27^2 + 175(103.5 - 55.27)^2 +$$
$$+ \frac{2}{4}[(103.5 - 55.27)^4 - 55.27^4] = -5237903 \text{ cm}^5$$

in which

$$A_{fo} = 140\ \text{cm}^{2.} \quad A_{fu} = 175\ \text{cm}^2 \text{are the upper and lower flange areas,}$$

$$I_y = 140 \cdot 55.27^2 + 175 \cdot (103.5 - 55.27)^2 + \frac{1}{12} \cdot 2 \cdot 100^3 + \\ + 2 \cdot 100(50 - 55.27)^2 = 1006963\ \text{cm}^4$$

and therefore,

$$r = -5237903/1006963 = -5.20\ \text{cm}$$

$$z_j = 13.18 - \frac{1}{2}(-5.20) = 15.78\ \text{cm}$$

d) Determination of auxiliary coefficients (see section 4.3, Table 4.4)

For a simply supported beam, it is $\psi = 1$. $\mu_o = 0.001$ and:

$$\mathrm{I} = \frac{1}{7} + \frac{\psi}{4.6} + \frac{\psi^2}{7} - \frac{1+\psi}{2.3\ \mu_o} + \frac{0.39}{\mu_o^2} = 389\,131$$

$$H = \frac{1+\psi}{4} - \frac{1}{4.3\ \mu_o} = -232$$

$$C_{1.B} = \frac{1}{\sqrt{2I}} = \frac{1}{\sqrt{2 \cdot 389131}} = 0.001133$$

$$C_1 = C_{1.\mathrm{B}}/\mu_o = 1.133$$

$$C_2 = -\frac{0.28658}{\mu_o\sqrt{I}} = -0.459$$

$$C_3 = H\sqrt{2}/\sqrt{1} = -0.526$$

e) Evaluation of the elastic critical moment

The elastic critical moment is calculated from the following formula:

$$M_{cr} = C_1 \frac{\pi^2 \mathrm{EI}_z}{(kl)^2} \left\{ \left[\frac{I_w}{I_z} + \frac{(kl)^2 G J_t}{\pi^2 \mathrm{EI}_z} + (\ C_2 z_g - C_3 z_j) \right]^{0.5} - (C_2 z_g - C_3 z_j) \right\}$$

in which, $C_{1.}$ $C_{2.}$ C_3 from paragraph (d), l is the span of the beam, $k = 1$ according to the supports' conditions (simple torsional restraints), I_z and z_j as determined in (b) and (c) correspondingly.

Moreover:

Warping constant

$$I_w = \frac{I_{fzo} I_{fzu}}{I_z} h^2 = 13232 \cdot 10^4\ \text{cm}^4$$

Torsion constant

$$J_t = \frac{1}{3}(50 \cdot 3.5^3 + 40 \cdot 3.5^3 + 100 \cdot 2^3) = 1553\ \text{cm}^4$$

Distance z_g

$$z_g = z_\alpha - z_s = 0 - 13.18 = -13.18\ \text{cm}$$

and finally:

$$M_{cr} = 180\,124\ \text{kN cm}$$

9.11.4 Design resistance moment for lateral-torsional buckling and verification of cross-section

Plastic section modulus

$$W_{pl,y} = 50 \cdot 3.5 \cdot 43 + 41.25^2 \cdot 2 \cdot \frac{1}{2} + 40 \cdot 3.5 \cdot 60.5 + 58.75^2 \cdot 2 \cdot \frac{1}{2} = 21148 \text{ cm}^3$$

Relative slenderness

$$\overline{\lambda}_{LT} = (W_{pl,y} \cdot f_y / M_{cr})^{0.5} = (21148 \cdot 35.5/180124)^{0.5} = 2.042$$ 6.3.2.2

The cross-section is welded and $h/b = 1070/400 = 2.68 > 2$, so the (d) buckling curve should be used.

The imperfection factor for buckling curve (d) is $\alpha_{LT} = 0.76$ so: Tab. 6.4

$$\Phi_{LT} = 0.5[1 + 0.76(2.042 - 0.20) + 2.042^2] = 3.285$$ Tab. 6.3

$$x_{LT} = \frac{1}{3.285 + (3.285^2 - 2.042^2)^{0.5}} = 0.170$$ Eq. 6.56

Design moment resistance:

$$M_{b,Rd} = x_{LT} W_{pl,y} f_y / \gamma_{M1} = 0.170 \cdot 21148 \cdot 35.5/1.0 \cdot 100 = 1276 \text{ kNm}$$ Eq. 6.55

Verification of capacity:

$$M_{b,Rd} = 1276 kNm > M_{Ed} = 392.7 \text{ kNm}$$

9.12 Example: Lateral-torsional buckling of a plate girder with an intermediate lateral restraint

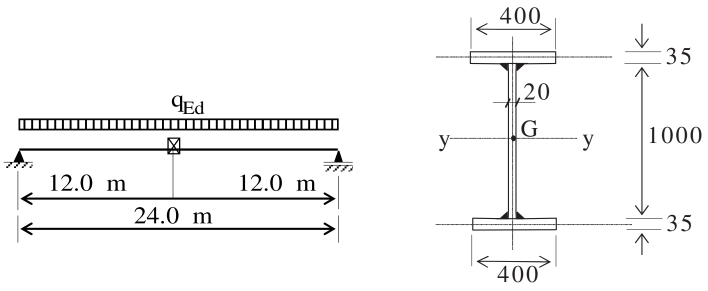
The simply supported plate girder shown in Fig. 9.21a, besides its self-weight, is subjected to the weight of a wet concrete with 20 cm thickness and 3.50 m width. The beam is laterally restrained at the supports and at the midspan before concreting. Verify the capacity of the girder (Fig. 9.21b) for lateral torsional buckling during the above stage of construction. The two supports are arranged with simple torsional (forked) restraints. Steel grade S 355.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.12.1 Design moment

Self-weight of steel beam

$$g = (2 \cdot 40 \cdot 3.5 + 100 \cdot 2) \cdot 7.85 \cdot 10^{-3} = 3.77 \text{ kN/m}$$



(a) (b)

Fig. 9.21. Simply supported plate girder with an intermediate lateral restraint

Self-weight of concrete

$$g_c = 0.20 \cdot 1.0 \cdot 3.50 \cdot 25 = 17.50\,\text{kN/m}$$

Total vertical design load

$$g_{Ed} = 1.35(3.77 + 17.50) = 28.72\,\text{kN/m}$$

Design moment

$$M_{Ed} = \frac{1}{8} \cdot g \cdot L^2 = \frac{1}{8} \cdot 28.72 \cdot 24.0^2 = 2068\,\text{kNm}$$

9.12.2 Point of application of vertical loads

The weight of the concrete is applied at the level of the upper flange while the self-weight of the steel beam is applied at the center of gravity of the cross-section. Therefore the total vertical load is applied at a distance z_α from the center of gravity, obtained through the equation:

$$17.50 \cdot 53.5 = (17.50 + 3.77) \cdot z_\alpha$$

and $z_\alpha = 44.0$ cm.

9.12.3 Elastic critical moment for lateral torsional buckling

The same formula as in Example 9.11 will be used. Since the cross-section is doubly symmetric:

$$z_s = 0 \quad \text{and} \quad z_j = 0.$$

The properties of the beam are: (Literature)

$$C_1 = 1.350. \quad C_2 = 0.0343$$
$$k = 0.50. \quad L = 24.0 \text{ m}$$
$$I_z = 2 \cdot \frac{1}{12} \cdot 3.5 \cdot 40^3 = 37333 \text{ cm}^4$$
$$I_w = \frac{1}{4} \cdot 37333 \cdot 103.5^2 = 9998 \cdot 10^4 \text{ cm}^6$$
$$J_t = \frac{1}{3}(2 \cdot 40 \cdot 3.5^3 + 100 \cdot 2^3) = 1410 \text{ cm}^4$$
$$z_g = z_\alpha - z_s = z_\alpha = 44.0 \text{ cm}$$

and therefore,

$$M_{cr} = 3722 \text{ kNm}$$

The cross-section is classified as class 1 (the calculations are omitted in this example) and thus:

$$W_{pl,y} = 2(40 \cdot 3.5 \cdot 51.75 + 50 \cdot 2 \cdot 25) = 19490 \text{ cm}^3$$
$$\overline{\lambda}_{LT} = (19490 \cdot 35.5/37.22 \cdot 100)^{0.5} = 1.364$$

The cross-section is welded with $h/b = 1070/400 = 2.68 > 2$. So, in order to calculate the reduction factor x , the buckling curve $d(\alpha_{LT} = 0.76)$ should be used. Tab. 6.4

$$\Phi_{LT} = 0.5[1 + (1.364 - 0.2) \cdot 0.76 + 1.364^2] = 1.872$$
$$\Phi_{LT} + (\Phi_{LT}^2 - \bar{\lambda}_{LT}^2)^{05} = 3.154 \quad \text{and}$$
$$x_{LT} = 1/3.154 = 0.317$$

The design resistance in lateral torsional buckling is then:

$$M_{b,Rd} = x_{LT} \cdot W_{pl,y} f_y / \gamma_{M1} = 0.317 \cdot 19490 \cdot 35.5/1.0100 = 2193 \text{ kNm} \quad \text{Eq. 6.55}$$

9.12.4 Verification of capacity

$$M_{Ed} = 2068 \text{ kNm} < M_{b,Rd} = 2193 \text{ kNm}$$

and thus, the beam is adequate for the stage of construction examined.

9.13 Example: Purlin without lateral restraint

Verify the adequacy of the purlin shown in Fig. 9.22 with an IPE 200 cross-section and steel grade S 235. The roof loading consists of permanent loads $g = 0.20 \text{ kN/m}^2$ and snow load $s = 0.40 \text{ kN/m}^2$. The span of the purlins is 7.2 m and the spacing of them is 4.0 m. The inclination of the roof is 10°. It is considered that the cladding does not provide any lateral restraint to the purlins.

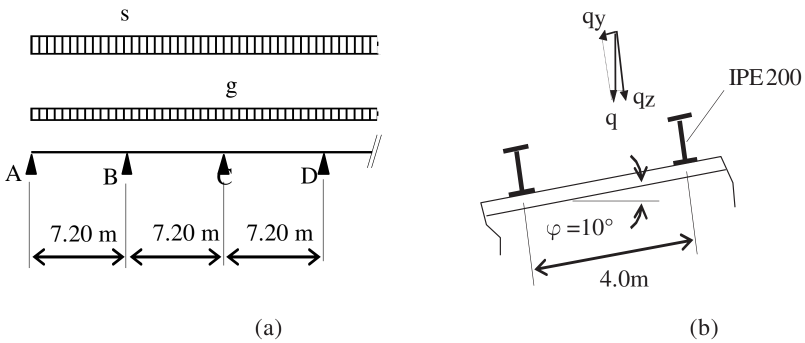


Fig. 9.22. Geometrical data and loading of the purlin

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

Geometrical properties of IPE 200

$$I_y = 1940\ \text{cm}^4,\quad I_z = 142\ \text{cm}^4,\quad W_y = 194\ \text{cm}^3,\quad W_z = 28.5\ \text{cm}^3,$$
$$i_z = 2.24\ \text{cm},\quad W_{pl,y} = 220\ \text{cm}^3,\quad I_w = 12990\ \text{cm}^6,\quad I_t = 7.02\ \text{cm}^4$$
$$t_f = 8.5\ \text{mm},\quad t_w = 5.6\ \text{mm},\quad b_f = 100\ \text{mm}.$$

9.13.1 Structural analysis

Design load:

$$p_d = 1.35g + 1.5s = 1.35 \cdot 0.20 + 1.5 \cdot 0.40 = 0.87\text{kN/m}^2$$ EN 1990

Design load of a purlin:

$$q_d = p_d \cdot 4 = 0.87 \cdot 4 = 3.48\ \text{kN/m}$$

Component in the direction of the web (Fig. 9.22b)

$$q_{zd} = q_d \cos\phi = 3.48 \cdot \cos 10^\circ = 3.43\ \text{kN/m}$$

Transversal component

$$q_{yd} = q_d \sin\phi = 3.48 \cdot \sin 10^\circ = 0.60\ \text{kN/m}$$

The cross-section belongs to class 1.

For the determination of the internal forces and moments elastic analysis is applied.

Maximum bending moments (support B):

$$M_{ySd} = q_{zd}\frac{l^2}{9.52} = 3.43 \cdot \frac{7.20^2}{9.52} = 18.7\ \text{kNm}$$
$$M_{zSd} = q_{yd}\frac{l^2}{9.52} = 0.60 \cdot \frac{7.20^2}{9.52} = 3.27\ \text{kNm}$$

Corresponding shear forces:

$$V_{z,Ed} = q_{zd}\frac{l}{2} + \frac{M_{y,Ed}}{l} = 3.43 \cdot \frac{7.20}{2} + \frac{18.7}{7.20} = 14.9\ \text{kN}$$

$$V_{y,Ed} = q_{yd}\frac{l}{2} + \frac{M_{z,Ed}}{l} = 0.60 \cdot \frac{7.20}{2} + \frac{3.27}{7.20} = 2.61\ \text{kN}$$

9.13.2 Verification for lateral torsional buckling

9.13.2.1 General

The verification should be performed in each span separately. In this particular case the span AB, which is the more sensitive in lateral torsional buckling due to the distribution of moments, will be examined. Thus, this span is extracted from the rest of the structure, and the moments calculated above as well as the intermediate transversal loading are applied. 5.2.2(7b)

9.13.2.2 Verification of span AB-General case 6.3.2.2

Ratio of moments in the segment AB:

$$\psi = \frac{M_A}{M_B} = 0$$

Transverse loading factor (see section 4.3, Table 4.4):

$$\mu_o = \frac{-M_B}{ql^2/8} = \frac{ql^2/9.52}{ql^2/8} = 0.84$$

For a member without any intermediate support

$$I = \frac{1}{7} + \frac{\psi}{4,6} + \frac{\psi^2}{7} - \frac{1+\psi}{2.3\,\mu_o} + \frac{0.39}{\mu_o^2} = 0.178 \quad k = 1$$

$$C_1 = \frac{1}{\sqrt{2 \cdot 0.178}} = 1.68$$

$$C_2 = \frac{0.28658}{\mu_o \sqrt{I}} = \frac{0.28658}{0.84\sqrt{0.178}} = 0.809$$

$C_3 = 0$ (doubly symmetric cross-section)

$z_g = 200/2 = 100\ \text{mm}$ (load is applied at the upper flange)

Critical moment for lateral torsional buckling:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kl)^2}\left[\sqrt{\frac{I_w}{I_z} + \frac{(kl)^2}{\pi^2} + \frac{GI_t}{EI_z} + (C_2 z_g)^2} - (C_2 z_g)\right] =$$

$$= 1.68 \cdot \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 142}{(1 \cdot 720)^2}\left[\sqrt{\frac{12990}{142} + \frac{(1720)^2}{\pi^2} + \frac{7.02}{2.6 \cdot 142} + (0.809 \cdot 10)^2} - \right.$$

$$\left. + (0.809 \cdot 10)\right] = 2471\ \text{kNcm} = 24.71\ \text{kNm}$$

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{194 \cdot 23{,}5}{2471}} = 1.36$$

Hot rolled section: h / b = 200 / 100 = 2 → Buckling curve a Tab. 6.4

$$\alpha_{LT} = 0.21$$ Tab. 6.3

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2] = 0.5[1 + 0.21(1.36 - 0.2) + 1.36^2] = 1.55$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + -\sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = \frac{1}{1.55 + \sqrt{1.55^2 - 1.36^2}} = 0.44$$ Eq. 6.56

Since no axial force is applied in the member, all k factors in eq. 6.61 and eq. 6.62 are equal to unity. Furthermore, the section is not of class 4. so all ΔM terms are equal to zero and the verification is performed through the interaction formula:

$$\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{MI}}} + \frac{M_{z,Ed}}{\frac{M_{zRk}}{\gamma_{MI}}} \leq 1$$ Eq. 6.61

where:

$$M_{y,Rk} = M_{pl,y} = W_{pl,y} \cdot f_y = 220 \cdot 23.5 = 5170 \text{ kNcm}$$

$$M_{z,Rk} = M_{pl,z} = 1.5W_{el,z} \cdot f_y = 1.5 \cdot 28.5 \cdot 23.5 = 1005 \text{ kNcm}$$ Tab. 6.7

The above formula finally leads to:

$$\frac{1870}{0.44 \cdot \frac{5170}{1.0}} + \frac{327}{\frac{1005}{1.0}} = 0.82 + 0.33 = 1.15 > 1.0$$

Therefore, the cross-section of the purlin is not sufficient against lateral torsional buckling and a stronger cross-section should be used or an intermediate lateral support might be arranged.

9.13.2.3 Verification of span AB using the method for rolled sections 6.3.2.3

Rolled section: $h/b = 2 \rightarrow$ Buckling curve b Tab. 6.5

$$\alpha_{LT} = 0.34$$ Tab. 6.3

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}) + \beta\bar{\lambda}_{LT}^2] =
= 0.5[1 + 0.34(1.36 - 0.4) + 0.75 \cdot 1.36^2] = 1.35$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} = \frac{1}{1.35 + \sqrt{1.35^2 - 0.75 \cdot 1.36^2}} = 0.50 \begin{array}{l} < 1.0 \\ < \frac{1}{1.36^2} = 0.54 \end{array}$$ Eq. 6.57

and:

$$\frac{1870}{0.50 \cdot 5170/1.0} + \frac{327}{1005/1.0} = 0.72 + 0.33 = 1.05 > 1.0$$

The section is still out (but close) of the limits.

9.13.3 Check of cross-section B

In addition to the above verification, a cross-section check at points A and B of the member should be performed. In this case the most unfavourable section is at support B. 6.3.3(2)

y-y axis

$$h_w = 200 - 2 \cdot 8.5 = 183 \text{ mm}$$

$$A_v = 28.5 - 2 \cdot 10 \cdot 0.85 + (0.56 + 2 \cdot 1.2) \cdot 0.85 = 14.0 \text{ cm}^2 > nh_w t_w =$$

$$= 1 \cdot 18.3 \cdot 0.56 = 10.3 \text{ cm}^2$$ 6.2.6(3)

$$V_{pl,z,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{M0}} = \frac{14.0 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 190 \text{ kN}$$ Eq. 6.18

$$V_{z,Ed} = 14.9 \text{ kN} < V_{pl,z,Rd} = 190 \text{ kN}$$

Besides, since $V_{z,Ed}/V_{pl,z,Rd} = 14.9/190 = 0.078 < 0.5$ no reduction of the moment resistance $M_{y,Rd}$ due to presence of shear forces is required. 6.2.8(2)

z-z axis

$$A_v = 2b_f t_f = 2 \cdot 10 \cdot 0.85 = 17.0 \text{ cm}^2$$ 6.2.6(3)

$$V_{pl,y,Rd} = \frac{17 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 230.7 \text{ kN}$$ Eq. 6.18

Besides, since $\frac{V_{y,Ed}}{V_{pl,y,Rd}} = \frac{2.61}{230.7} = 0.01 < 0.5$ no reduction of the moment $M_{z,Rd}$ due to presence of shear forces is required.

Check in biaxial bending $(n = 0)$

Since $\gamma_{M0} = \gamma_{M1} = 1.0$ and no axial force is applied $(n = 0)$:

$$M_{N,y,Rd} = M_{c,y,Rd} = 5170 \text{ kNcm}$$

$$M_{N,z,Rd} = M_{c,z,Rd} = 1005 \text{ kNcm}$$

$$\alpha = 2 \qquad \beta = 5 \cdot 0 = 0$$

but it is taken $\beta = 1$:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} = \left(\frac{18.7}{51.70}\right)^2 + \left(\frac{3.27}{10.50}\right)^1 = 0.13 + 0.33 = 0.46 < 1$$

9.14 Example: Purlin laterally restrained

Verify the capacity of the purlin shown in Fig. 9.23 with an IPE 200 cross-section and steel grade S 235. The design load of the roof is $q = 1.40 \text{ kN/m}^2$. The span of the purlins is 7.2 m and the spacing between them is 4.0 m. The existing continuous trapezoidal steel sheeting has an effective second moment of area $I_{\text{eff}} = 90 \text{ cm}^4/\text{m}$ and provides lateral restraint to the purlins.

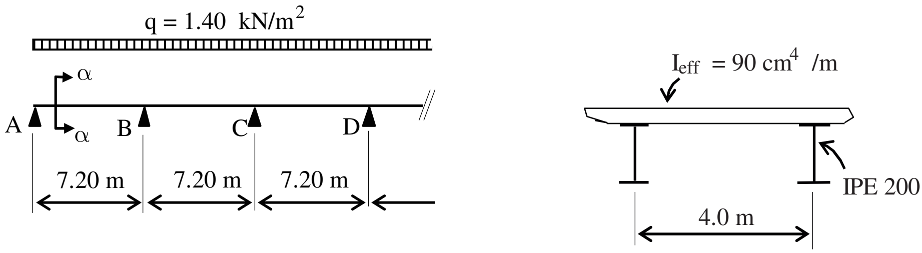


Fig. 9.23. Structural system and loading of the purlins

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

Geometrical properties of IPE 200

$$
\begin{array}{llll}
I_y = 1940\ \text{cm}^4, & I_z = 142\ \text{cm}^4, & W_y = 194\ \text{cm}^3, & W_z = 28.5\ \text{cm}^3, \\
i_z = 2.24\ \text{cm}, & W_{pl,y} = 220\ \text{cm}^3, & I_w = 12990\ \text{cm}^6, & I_t = 7.02\ \text{cm}^4 \\
t_f = 8.5\ \text{mm}, & t_w = 5.6\ \text{mm}, & b_f = 100\ \text{mm} &
\end{array}
$$

9.14.1 Structural analysis

Due to the lateral restraint, the purlin is loaded only parallel to its web, irrespectively of the roof inclination. The loading component parallel to the flanges is resisted by the roof's sheeting, acting as diaphragm.

Design load of the purlin

$$q_d = 1.40 \cdot 4 = 5.60\ \text{kN/m}$$

The cross-section is of class 1.

For the determination of the internal forces and moments elastic analysis is applied.

Bending moment at support B (maximum moment):

$$M_B = \frac{q_d l^2}{9.52} = \frac{5.6 \cdot 7.20^2}{9.52} = 30.5\ \text{kNm}$$

Corresponding shear force:

$$V_B = \frac{q_d l}{2} + \frac{M_B}{l} = \frac{5.6 \cdot 7.2}{2} + \frac{30.5}{7.20} = 24.4\ \text{kN}$$

9.14.2 Verification of span AB to lateral torsional buckling

The roof steel sheeting provides translational restraint to the upper flange and rotational restraint to the purlin.

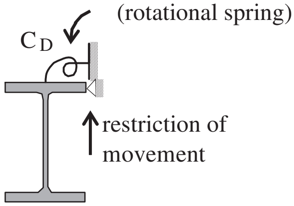


Fig. 9.24. Modelling of the purlin's lateral restraint

The static model of the system is shown in Fig. 9.24.
Spring stiffness:

$$\frac{1}{C_\Theta} = \frac{1}{C_{D,A}} + \frac{1}{C_{D,C}}$$ EN 1993-1-3. Eq. 10.14

where:

$$C_{D,C} = k\frac{EI_{\text{eff}}}{s}$$ (rotational stiffness corresponding to the flexural stiffness of the sheeting) EN 1993-1-3. Eq. 10.16

$k = 4$ continuous sheeting

$s = 4.0$ m spacing of the purlins

and thus:

$$C_{D,C} = 4 \cdot \frac{2.1 \cdot 10^4 90/100}{400} = 189\ \text{kN}$$

$$C_{DA} = C_{100} \cdot k_{b\alpha} \cdot k_t \cdot k_{br} \cdot k_A \cdot k_{bT}$$ EN 1993-1-3. Eq. 10.17

Negative positioning of sheeting, sheet fastened through the crest and pitch of fasteners $e = b_f$.

Thus: $C_{100} = 10$ kN and $b_{\text{T,max}} = 40$ mm EN 1993-1-3. Tab. 10.3. line 3

Width flange of the purlin:

$$b_\alpha = 100\ \text{mm} < 125\ \text{mm} \rightarrow k_{b\alpha} = (b_\alpha/100)^2 = 1$$

Thickness of the sheeting:

$$t_{\text{nom}} = 0.65\ \text{mm} \rightarrow k_t = \left(\frac{t_{\text{nom}}}{0.75}\right)^{1,5} = 0.80$$

Corrugation width:

$$b_R = 160\ \text{mm} < 185\ \text{mm} \rightarrow k_{br} = 1$$

Loading transferred from the sheeting to the purlin:

$$A = q = 5.6\ \text{kN/m}$$

For gravity loading, $t_{nom} = 0.75$ mm, negative positioning

$k_A = 1.0 + (A-1)\cdot 0.16 = 1 + (5.6-1)\cdot 0.16 = 1.74$

$b_T = 75$ mm (width of the sheeting flange through which it is fastened to the purlin)

$$k_{bT} = \sqrt{\frac{b_{T,\max}}{b_T}} = \sqrt{\frac{40}{75}} = 0.73$$

Thus,

$$C_{DA} = 10\cdot 1\cdot 0.80\cdot 1\cdot 1.74\cdot 0.73 = 10.2\ \text{kN}$$

$$\frac{1}{C_\theta} = \frac{1}{10.2} + \frac{1}{189} \rightarrow C_\theta = 9.68\ \text{kN}$$

Critical moment for lateral torsional buckling:

$$\delta = \frac{\sqrt{C_\theta\cdot EI_w}}{GI_t} = \frac{\sqrt{9.68\cdot 2.1\cdot 10^4\cdot 12990}}{\frac{2.1\cdot 10^4}{2.6}\cdot 7.02} = 0.906$$

$$\alpha_T = \frac{l^2 GI_t}{\pi^2 EI_w} = \frac{720^2\cdot 7.02}{\pi^2\cdot 2.6\cdot 12990} = 10.9 > 10$$

Moments ratio $\psi = \frac{M_A}{M_B} = 0$ Example 9.13

Transverse loading factor $\mu_o = 0.84$ Example 9.13

$$\alpha = 15\mu_o^{-1.4} = 15\cdot 0.84^{-1.4} = 19.1$$

$$b = 1.5\mu_o^{-0.26} = 1.5\cdot 0.84^{-0.26} = 1.57$$

$$c = 3.5\mu_o^{-0.18} = 3.5\cdot 0.84^{-0.18} = 3.61$$

$$d = 9.4\mu_o^{-0.73} = 9.4\cdot 0.84^{-0.73} = 10.7$$

$$k_w = \alpha + b\cdot\alpha_T + c\cdot\alpha_T\cdot\delta + d\cdot\delta =$$
$$= 19.1 + 1.57\cdot 10.9 + 3.61\cdot 10.9\cdot 0.906 + 10.7\cdot 0.906 = 81.6$$

$$M_{cr} = \frac{\pi^2 EI_w}{l^2(h-t_f)}k_w = \frac{\pi^2\cdot 2.1\cdot 10^4\cdot 12990}{720^2(20-0.85)}\cdot 81.6 = 22130\ \text{kNcm}$$

The critical moment is increased analogously to Example 9.13. due to the existence of the diaphragm of the roof.

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{230\cdot 23.5}{22130}} = 0.49 \qquad 6.3.2.2$$

Using the general method as in Example 9.13. we find:

$$\Phi_{LT} = 0.5[1 + 0.21(0.49-0.2) + 0.49^2] = 0.65 \qquad \text{Eq. 6.56}$$

$$\chi_{LT} = \frac{1}{0.65 + \sqrt{0.65^2 - 0.49^2}} = 0.93 \qquad \text{Eq. 6.55}$$

$$M_{b,Rd} = x_{LT} W_y \frac{f_y}{\gamma_{M1}} = 0.93\cdot 230\cdot\frac{23.5}{1.0} = 5026\ \text{kNcm} \qquad \text{Eq. 6.54}$$

$$\text{Ε}\lambda\varepsilon\gamma\chi o\varsigma : \frac{M_{Ed}}{M_{b,Rd}} = \frac{30.5}{50.26} = 0.61 < 1$$

The section is adequate.

The verification of the rest of the purlins (BC, CD, ...) might be performed using the same procedure. It is obvious that in this example the member AB is the most critical against lateral torsional buckling.

9.14.3 Check of cross-section B

In addition to the above verification, a cross-section check at points A and B of the member should be performed. In this case the most unfavourable section is at support B. 5.4.5.2

Check in bending

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{Mo}} = \frac{220 \cdot 23.5}{1.0} = 5170 > M_{Ed} = 3050 \text{ kNcm}$$

Check in shear

$$A_v = 14.0 \text{ cm}^2$$

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{Mo}} = 14.0 \cdot \frac{23.5}{\sqrt{3}} \cdot \frac{1}{1.00} = 190 \text{ kN}$$ Example 9.13

$$V_{Ed} = 24.4 \text{ kN} < V_{pl,z,Rd} = 190 \text{ kN}$$

Check in bending and shear

Since:

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{24,4}{190} = 0.13 < 0.5$$

check for this combination is not required. 6.2.8(2)

9.15 Example: Column under axial compressive load

Verify the adequacy of the 8 m column shown in Fig. 9.25. belonging to a non sway storey, with a cross-section of HEB 300 and subjected only to a design axial compressive force $N_{Ed} = 2000$ kN. The lower end of the column is pinned in both directions, while the upper end is non-sway, pinned for buckling about y-y axis and fixed for buckling about z-z axis. Steel grade S 235.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

The following data apply:

$$f_y = 235 \text{ N/mm}^2 = 23.5 \text{ kN/cm}^2$$ Tab. 3.1

$$A = 149 \text{ cm}^2$$

buckling length coefficients $k_y = 1$

$$k_z = 0.7$$

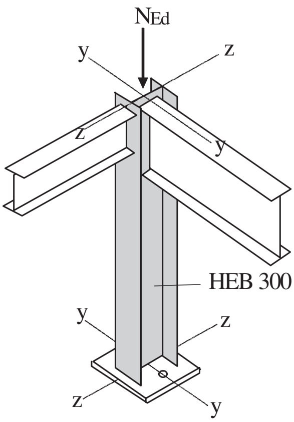


Fig. 9.25. Column under axial compressive load

9.15.1 Cross-section classification

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/235} = 1$$ Tab. 5.2

Flange: Tab. 5.2. sheet 2

$$\frac{c}{t} = \frac{150 - 5.5 - 27}{19} = 6.18 < 9\varepsilon = 9$$

The flange belongs to class 1
Web: Tab. 5.2. sheet 1

$$\frac{c}{t} = \frac{300 - 2(19 + 27)}{11} = \frac{208}{11} = 18.91 < 33\varepsilon = 33$$

The web belongs to class 1 and the whole cross-section to class 1. 5.5.2(6)

9.15.2 Verification

The column is adequate if the following relation is satisfied in both principal axes of the cross-section:

$$N_{Ed} \leqslant N_{b,Rd} = \chi A f_y / \gamma_{M1} \quad \text{(class 1 cross-section)}$$ Eq. 6.47

where $\gamma_{M1} = 1.0$ 6.1

9.15.2.1 Buckling verification about y-y axis

It is:

$$\frac{h}{b} = \frac{300}{300} = 1 < 1.2$$ Tab. 6.2

$$t_f = 19\text{mm} < 100\text{mm}$$

Thus, for S 235 the relevant buckling curve is *b*

$$\lambda_y = \frac{k_y L_y}{i_y} = \frac{1 \cdot 800}{13} = 61.54$$ 6.3.1.3(1)

$$\lambda_1 = 93.9\varepsilon = 93.9$$

$$\overline{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{61.54}{93.9} = 0.655 \quad \text{(relative slenderness)}$$ Eq. 6.50

And

$$\chi = 0.81 \quad \text{(reduction factor)}$$ Fig. 6.4

The reduction factor could be calculated analytically as following:

$$\Phi = \left[1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right] = \\ = 0.[1 + 0.34 \times (0.655 - 0.2) + 0.655^2] = 0.792$$ 6.3.1.2

$$\chi = 1 \Big/ \left[\Phi + \left(\Phi^2 - \overline{\lambda}^2\right)^{0.5}\right] = 0.8082 < 1$$ Eq. 6.49

And therefore:

$$N_{b.Rd} = 0.8082 \cdot 149 \cdot 23.5/1.0 = 2830\,\text{kN} > N_{Ed} = 2000\,\text{kN}$$

9.15.2.2 Buckling verification about z-z axis

For $\frac{h}{b} = \frac{300}{300} = 1 < 1.2$ and $t_f = 19$ mm < 100 mm, the buckling curve is c, Tab. 6.2

$$\lambda_z = \frac{k_z L_z}{i_z} = \frac{0.7 \cdot 800}{7.58} = 73.88$$

$$\lambda_1 = 93.9$$

$$\overline{\lambda}_z = \frac{73.88}{93.9} = 0.787$$

$$\Phi = 0.5[1 + 0.49 \cdot (0.787 - 0.2) + 0.787^2] = 0.953$$ Eq. 6.49

$$\chi = 1/[0.953 + (0.953^2 - 0.787^2)^{0.5}] = 0.671$$

and therefore:

$$N_{b.Rd} = 0.671 \cdot 149 \cdot 23.5/1.0 = 2350\,\text{kN} > \text{N}_{\text{Ed}} = 2000\,\text{kN}$$

9.16 Example: Column under axial compressive load with intermediate lateral supports

The maximum design axial compressive load N_{Ed} of a column with an IPE 300 cross-section and a height of 7.50 m (Fig. 9.26) is required. The two ends of the column have not the possibility of a relative movement in both directions and they are pinned for buckling about z-z axis and fixed for buckling about y-y axis. The horizontal intermediate supports, which might be side rails, ensure a lateral support of the column in the thirds of its height (Fig. 9.26). Steel grade S 275.

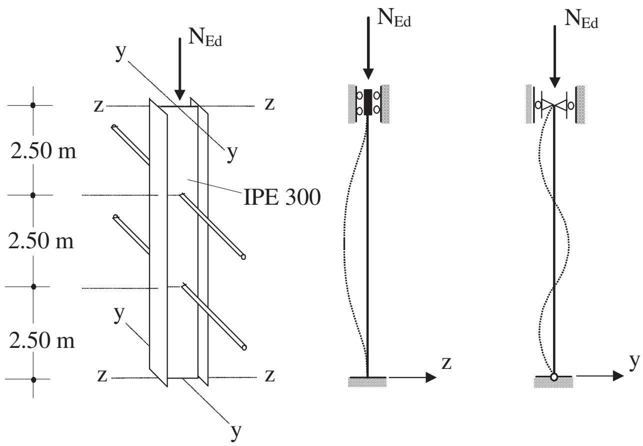


Fig. 9.26. Support conditions of the column

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

The following data apply:

$$f_y = 275\ \text{N/mm}^2$$ Tab. 3.1

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/275} = 0.92 A = 53.8\ \text{cm}^2$$ Tab. 5.2

Buckling lengths:

About y-y axis $$l_y = \frac{7.50}{2} = 3.75m$$

About z-z axis $$l_z = \frac{7.50}{3} = 2.50\ \text{m}$$

9.16.1 Cross-section classification

Tab. 5.2

Flange:

$$\frac{c}{t} = \frac{75 - 3.55 - 15}{10.7} = 5.28 < 9\varepsilon = 9 \cdot 0.92 = 8.28$$

The flange belongs to class 1
Web:

$$\frac{c}{t} = \frac{300 - 2 \cdot (10.7 + 15)}{7.1} = 35.0 > 33\varepsilon = 33 \cdot 0.92 = 30.4$$

but $\leqslant 38\varepsilon = 38 \cdot 0.92 = 35$.

The web belongs to class 2 and thus the whole cross-section to class 2. 5.5.2(6)

9.16.2 Verification

The column is adequate if the following relation is satisfied in both principal axes of the cross-section:

$$N_{Ed} \leqslant N_{b,Rd} = \chi A f_y / \gamma_{M1} \quad \text{(class 2 cross-section)}$$ Eq. 6.47

where

$$\gamma_{M1} = 1.0$$ 6.1

9.16.2.1 *Resistance for buckling about y-y axis*

$$\frac{h}{b} = \frac{300}{150} = 2 > 1.2$$ Tab. 6.2

$$t_f = 10.7 \text{ mm} < 40 \text{ mm}$$

Thus, for S 275 the relevant buckling curve is a,

$$\lambda_y = \frac{l_y}{i_y} = \frac{375}{12.5} = 30$$

$$\lambda_1 = 93.9\varepsilon = 93.9 \cdot 0.92 = 86.4$$

$$\overline{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{30}{86.4} = 0.347 \quad \text{(relative slenderness)}$$ Eq. 6.50

and

$$\chi_y = 0.966 \quad \text{(reduction factor)}$$ Fig. 6.4

Therefore:

$$\max N_{Ed} = N_{b.Rd} = 0.966 \cdot 53.8 \cdot 27.5/1.0 = 1429 \text{ kN}$$

9.16.2.2 *Resistance for buckling about z-z axis*

For

$$\frac{h}{b} = \frac{300}{150} = 2 > 1.2$$

$$t_f = 10.7 \text{ mm} < 40 \text{ mm}$$

the relevant buckling curve is *b*, Tab. 6.2

$$\lambda_z = \frac{l_z}{i_z} = \frac{250}{3.35} = 74.6$$
$$\lambda_1 = 86.4$$
$$\overline{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{74.6}{86.4} = 0.863$$

and $\chi_z = 0.685$

Therefore:

$$\max N_{Ed} = 0.685 \cdot 53.8 \cdot 27.5/1.0 = 1013 \text{ kN}$$

and the maximum design axial compressive load N_{Ed} that could be applied to the column shown in Fig. 9.26 is:

$$N_{Ed} = 1013 \text{ kN}$$

9.17 Example: Buckling length of columns in a single storey frame

The buckling lengths of the columns in the frame shown in Fig. 9.27 are to be determined. The cross-section of the columns is HEB 220 and of the beams IPE 330. Lateral supports at the top of the columns prevent the out of plane movements, while the nodes in this direction are assumed as pinned. The supports of the columns are assumed as having the same conditions in both directions (i.e. fixed for A and C, and pinned for E). It is assumed that due to the external loading the beams are subjected to axial compressive forces equal to $0.3N_E$ (N_E is the Euler's elastic load).

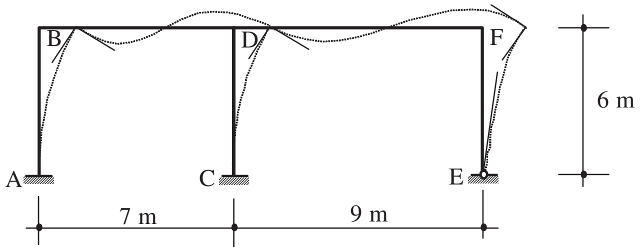


Fig. 9.27. Geometrical properties of the frame

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

The following data apply:

Columns:	HEB220	$I_y = 8090 \text{ cm}^4$
		$I_z = 2840 \text{ cm}^4$
Beams:	IPE330	$I_y = 11770 \text{ cm}^4$
		$I_z = 788 \text{ cm}^4$

9.17.1 Buckling lengths in the plane of the frame

The frame in its own plane is considered as sway and the corresponding deformed shape is indicated in Fig. 9.27.

The stiffness coefficients of the beams are:

$$K_{\mathrm{BD}} = 1.5\frac{I}{L}\left(1-0.2\cdot\frac{N}{N_E}\right) = 1.5\frac{11770}{700}(1-0.2\cdot 0.3) = 23.71 \qquad \text{(Literature)}$$

$$K_{\mathrm{DF}} = 1.5\frac{11770}{900}(1-0.2\cdot 0.3) = 18.44$$

The stiffness coefficients of the columns are:

$$K_{\mathrm{AB}} = K_{\mathrm{CD}} = K_{\mathrm{EF}} = \frac{8090}{600} = 13.48$$

and therefore, the distribution factors are calculated (ref. [4.54]): Column AB

$$n_1 = n_A = 0 \quad \text{(fixed)}$$

$$n_2 = \frac{13.48}{13.48+23.71} = 0.362$$

Column CD

$$n_1 = n_C = 0 \quad \text{(fixed)}$$

$$n_2 = \frac{13.48}{13.48+23.71+18.44} = 0.242$$

Column EF

$$n_1 = n_E = 1 \quad \text{(pinned)}$$

$$n_2 = \frac{13.48}{13.48+18.44} = 0.422$$

Based on these contribution factors, the equivalent buckling length coefficients and the corresponding buckling lengths of the columns are determined.

$$k_{y,\mathrm{AB}} = 1.14, \quad \lambda_{y,\mathrm{AB}} = 1.14\cdot 6 = 6.84\ \mathrm{m}$$

$$k_{y,\mathrm{CD}} = 1.08, \quad \lambda_{y,\mathrm{CD}} = 1.08\cdot 6 = 6.48\ \mathrm{m}$$

$$k_{y,\mathrm{EF}} = 2.35, \quad \lambda_{y,\mathrm{EF}} = 2.35\cdot 6 = 14.10\ \mathrm{m}$$

9.17.2 Buckling lengths out of the plane of the frame

The frame is in this direction non-sway and the corresponding equivalent buckling length coefficients (see section 7.2.2) and the buckling lengths are obtained as follows:

$$k_{z,\mathrm{AB}} = k_{z,\mathrm{CD}} = 0.70 \quad \text{(pinned-fixed column)}$$

$$k_{z,\mathrm{EF}} = 1.0 \quad \text{(pinned-pinned column)}$$

and

$$\lambda_{\mathrm{Z,AB}} = \lambda_{\mathrm{Z,CD}} = 0.7\cdot 6 = 4.20\ \mathrm{m}$$

$$\lambda_{\mathrm{Z,EF}} = 1\cdot 6 = 6\ \mathrm{m}.$$

9.18 Example: Buckling of a column of a multi-storey building

The maximum uniformly distributed load q_{Ed} on the beams of the frames shown in Figs. 9.28a, b is to be determined, uzing as criterion the buckling strength of the column (1-2) in the plane of the frames. The axial forces on the beams could be neglected. Steel grade S 235.

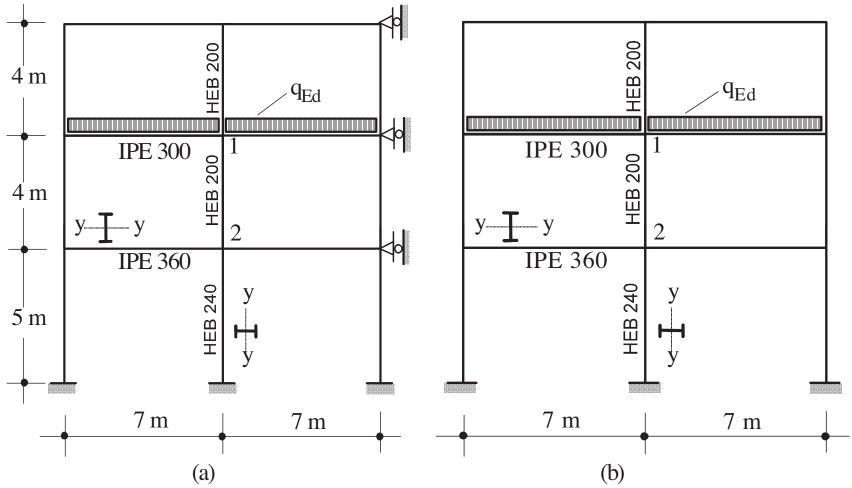


Fig. 9.28. Non-sway (a) and sway frame (b)

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

The following data apply:

Columns:	HEB200 :	$I_y = 5700\ \text{cm}^4$
	HEB 240:	$I_y = 11260\ \text{cm}^4$
Beams:	*IPE*300 :	$I_y = 8360\ \text{cm}^4$
	IPE360 :	$I_y = 16270\ \text{cm}^4$

9.18.1 Design axial compressive load

Due to the symmetry of the frame and the loading, the column (1-2) is subjected only to an axial load which could be approximately taken equal to:

$$N_{Ed} = 7q_{Ed}$$

(the exact value could be calculated from the frame analysis).

The buckling verification is written as: $N_{Ed} \leqslant N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ Eq. 6.47

where:

$$f_y = 23.5\ \mathrm{kN/cm^2}$$ Tab. 3.1

$$A = 78.1\mathrm{cm}^2, \qquad \gamma_{M1} = 1.0$$ 6.1

9.18.2 Cross-section classification

Tab. 5.2

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/235} = 1$$

Flange:

$$\frac{c}{t} = \frac{100 - 4.5 - 18}{15} = 5.17 < 9\varepsilon = 9 \quad \text{(flange under compression)}$$

The flange belongs to class 1

Web:

$$\frac{c}{t} = \frac{200 - 2 \cdot (15 + 18)}{9} = 14.89 < 33\varepsilon = 33 \quad \text{(web under compression)}$$

The web is class 1 and thus the whole cross-section belongs to class 1.

9.18.3 Non-sway frame (Fig. 9.28a)

Contribution factors η_1 and η_2 (ref. [4.54]).

It is assumed that the axial forces on the beams connected to 1 and 2 joints of the frame could be neglected, while their far end is considered as fully fixed. The effective stiffness of these beams is given by the formula:

$$k_{ij} = \frac{I}{L}$$

Thus:

$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} = \frac{2\frac{5700}{400}}{2\frac{5700}{400} + 2\frac{8360}{700}} = 0.544$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}} = \frac{\frac{5700}{400} + \frac{11260}{500}}{\frac{5700}{400} + \frac{11260}{500} + 2\frac{16270}{700}} = 0.442$$

and $k_y = 0.69$ (see section 7.2.2)

The value of k_y, could be calculated using the approximative equation (7.3) taken from ref. [4.54]:

$$k_y = 0.5 + 0.14(0.544 + 0.442) + 0.055(0.544 + 0.442)^2 = 0.692$$

Relative slenderness:

$$\lambda_y = \frac{k_y L_y}{i_y} = \frac{0.69 \cdot 400}{8.54} = 32.3$$ 6.3.1.3(1)

$$\lambda_1 = 93.9$$

$$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{32.3}{93.9} = 0.344$$ Eq. 6.50

It is:

$$\frac{h}{b} = \frac{200}{200} = 1 < 1.2$$ Tab. 6.2

$$t_f = 15 \text{ mm} < 100\text{mm}$$

For S 235 the relevant buckling curve is b Tab. 6.2
and $\chi = 0.947$ Fig. 6.4
Thus

$$N_{b,Rd} = 0.947 \cdot 78.1 \cdot 23.5/1.0 = 1738 \text{ kN}$$

$$7q \leqslant 1738$$

and max $q = 248.3$ kN/m

9.18.4 Sway frame (Fig. 9.28b)

$$\eta_1 = 0.544 \quad \text{(as previously)}$$

$$\eta_2 = 0.442$$

$$k_y = 1.47$$

The value of k_y, could be calculated using the approximative equation (7.4):

$$k_y = \left[\frac{1 - 0.2 \cdot (0.544 + 0.442) - 0.12 \cdot 0.544 \cdot 0.442}{1 - 0.8 \cdot (0.544 + 0.442) + 0.6 \cdot 0.544 \cdot 0.442}\right]^{0.5} = 1.47$$

$$\lambda_y = \frac{k_y L_y}{i_y} = \frac{1.47 \cdot 400}{8.54} = 68.85$$

$$\lambda_1 = 93.9$$

$$\bar{\lambda}_y = \frac{68.85}{93.9} = 0.733$$

For $\frac{h}{b} = 1 < 1.2$ and $t_f = 15$ mm < 100 mm the buckling curve is b so $\chi = 0.764$

$$N_{b,Rd} = 0.764 \cdot 78.1 \cdot 23.5/1.0 = 1402 \text{ kN}$$

and max $q = \frac{1402}{7} = 200.3$ kN/m

9.19 Example: Laterally restrained beam under compression and bending

The simply supported beam shown in Fig. 9.29 is to be verified. The beam is continuously laterally supported. The cross-section is IPE 300. the steel grade is S 235 and the design loads are $q_{Ed} = 10kN/m, P_{Ed} = 15$ kN and $N_{Ed} = 273$ kN.
Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

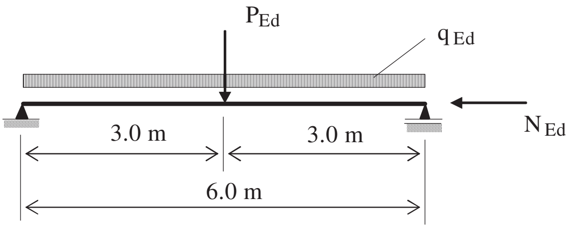


Fig. 9.29. Beam under compression and bending

9.19.1 Cross-section classification

Tab. 5.2

Steel grade S 235, so $\varepsilon = 1.0$

Flange:

$$c/t_f = \left(\frac{150-7.1}{2} - 15\right)/10.7 = 5.3 < 9, \quad \text{class 1}$$

Web:

In case of full plastification of the cross-section, the height of the plastified area of the web is $e = 273/0.71 \cdot 23.5 = 16.36$ cm, $c = 248$ mm

$$a_c = 124 + \frac{1}{2} \cdot 163.6 = 205.8 \text{ mm}$$

and $\alpha = 205.8/248 = 0.83 > 0.50$.

It is

$$c/t = 248/7.1 = 34.9 < 396\varepsilon/(13\alpha - 1) = 396/(13 \cdot 0.83 - 1) = 40.4$$

So, the web is class 1 and the whole cross-section class 1.

9.19.2 Cross-section verification

a) Verification in bending

The check is performed at midspan.

Design bending moment:

$$M_{Ed} = \frac{1}{8} \cdot 10 \cdot 6.0^2 + \frac{1}{4} \cdot 15 \cdot 6.0 = 67.5 \text{ kNm}$$

Design plastic moment of the section:

$$M_{pl.y.Rd} = W_{pl} f_y/\gamma_{M0} = 629 \cdot 23.5/1.0 = 14781 \text{ kNcm} = 147.8 \text{ kNm}$$ Eq. 6.13

Reduced design plastic resistance moment due to the axial force:

It is:

$$N_{pl.Rd} = A f_y/\gamma_{M0} = 53.8 \cdot 23.5/1.0 = 1264 \text{ kN}$$ Eq. 6.6

Since

$$N_{Ed} = 273\text{ kN} > 0.5h_w t_w f_y/\,\gamma_{M0} = \\ = 0.5(30 - 2\cdot 1.07)\cdot 0.71\cdot 23.5/1.0 = \\ = 232.4\text{ kN}$$ Eq. 6.34

The influence of the axial force on the design plastic moment should be considered.

$$n = N_{Ed}/N_{pl.Rd} = 273/1264 = 0.216$$ 6.2.9.1(5)

$$\alpha = \frac{A - 2bt_f}{A} = \frac{53.8 - 2\cdot 15\cdot 1.07}{53.8} = 0.403 < 0.50$$

and therefore,

$$M_{N.y.Rd} = M_{pl.y.Rd}(1-n)/(1-0.5\alpha) = \\ = 147.8\cdot(1-0.216)/(1-0.5\cdot 0.403) = \\ = 145.1 kNm > M_{Ed} = 67.5\text{ kNm}$$ Eq. 6.36

b) Verification in shear 6.26

The check should be performed at the support.

$$V_{Ed} = \frac{1}{2}\cdot 10\cdot 6.0 + \frac{1}{2}\cdot 15 = 37.5\text{ kN}$$

$$A_v = 53.8 - 2\cdot 15.0\cdot 1.07 + (0.71 + 2\cdot 1.5)\cdot 1.07 = 25.7\text{ cm}^2$$ 6.2.6(3)

$$V_{pl.Rd} = A_v\cdot(f_y/\sqrt{3})/\gamma_{M0} = 25.7\cdot 23.5/(1.0\cdot\sqrt{3}) = \\ = 348.7\text{ kN} > V_{Ed} = 37.5\text{ kN}$$ Eq. 6.18

Besides, at the position of maximum moment it is:

$$V_{Ed} = 37.5 - 3\cdot 10 = 7.5\text{ kN} < 0.50V_{pl.Rd} = 174.3\text{ kN}$$

So, a reduction of the moment due to shear force is not required. 6.2.8(2)

9.19.3 Member verification

Slenderness to y-y and z-z axes:

$$\lambda_y = 600/12.5 = 48$$

$$\bar{\lambda}_y = \lambda_y/(\pi\sqrt{E/f_y}) = 48/(\pi\sqrt{21000/23.5}) = 0.511$$

Since

$$h/b = 300/150 = 2 > 1.2 \quad \text{and} \quad t_f = 10.7 < 40\text{ mm},$$

in this case the corresponding buckling curve is a, and the imperfection factor is: Tab. 6.2

$$\alpha = 0.21$$ Tab. 6.1

$$\Phi = 0.5[1 + 0.21(0.511 - 0.20) + 0.511^2] = 0.663$$ Eq. 6.49

$$\Phi + (\Phi^2 - \bar{\lambda}_y^2)^{0.5} = 0.663 + (0.663^2 - 0.511^2)^{0.5} = 1.086$$

while the reduction factor x_y = 1/1.086 = 0.921.

Since the beam is laterally continuously restrained:

$$\lambda_z = 0 \quad \text{and} \quad x_z = 1.0.$$

For the verification of the member under compression and bending, the interaction formulae of EN 1993-1-1 paragraph 6.3.3 should be applied. Formula (6.6.1) with the aid of Tab. 6.7 in this example (class 1 cross-section) leads to:

$$\frac{N_{Ed}}{\frac{x_y A f_y}{\gamma M1}} + k_{yy} \frac{M_{y \cdot Ed}}{\frac{W_{pl \cdot y} f_y}{\gamma_{M1}}} \leqslant 1$$

Since the beam is laterally continuously restrained, the reduction factor due to lateral torsional buckling is taken as $x_{LT} = 1.0$. The coefficient k_{yy} is calculated using the relations of Tables A1 and A2 (Annex A).

$$N_{cr,y} = \pi^2 E I_y / l^2 = \pi^2 \cdot 21000 \cdot 8360 / 600^2 = 4813 \text{ kN}$$

$$C_{my} = C_{my,o} = 1 + 0.03 \cdot \frac{N_{Ed}}{N_{cr,y}} = 1 + 0.03 \frac{273}{4813} = 1.002$$ Annex A, Tab. A.2

$$\bar{\lambda}_o = b_{LT} = 0, \quad n_{p\lambda} = 0.216, \quad \bar{\lambda}_{\max} = 0.511$$

$$w_y = W_{p\lambda \cdot y} / W_{el \cdot y} = 629/557 = 1.129 < 1.50$$ Annex A, Tab. A.1

$$C_{yy} = 1 + (1.129 - 1)\left(2 - \frac{1.6}{1.129} \cdot 1.002^2 \cdot 0.511 - \frac{1.6}{1.129} \cdot 1.002^2 \cdot 0.511^2\right) \cdot$$
$$\cdot 0.216 = 1.025$$

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - x_y \frac{N_{Ed}}{N_{cr,y}}} = \frac{1 - \frac{273}{4813}}{1 - 0.921 \frac{273}{4813}} = 0.995$$

$$C_{mLT} = 1.0$$

$$k_{yy} = C_{my} \cdot \frac{1}{C_{yy}} = 1.002 \cdot \frac{1}{1{,}025} = 1.031$$

and therefore, the interaction formula becomes:

$$\frac{\frac{273}{0.921 \cdot 53.8 \cdot 23.5}}{1.0} + 1.031 \frac{\frac{6750}{629 \cdot 23.5}}{1.0} = 0.235 + 0.471 = 0.706 < 1$$

The member under compression and bending is sufficient for flexural buckling.

Remark 11. In case that no lateral support exists, the beam should be examined for lateral torsional buckling according to the interaction formulae 6.61 and 6.62.

9.20 Example: Flexural and lateral torsional buckling of a column

Verify the capacity of a simply supported column without any intermediate lateral restraint, subjected to a design axial compressive force N_{Ed} = 200 kN and a design bending moment $M_{y,Ed} = 10$ kNm applied at the top of the column about y-y axis. The cross-section is HEA 160 and the steel grade S 235.

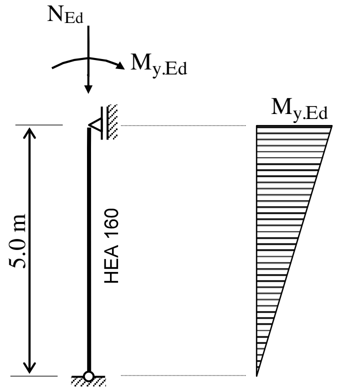


Fig. 9.30. Simply supported column

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.20.1 Geometrical properties of the cross-section

$$\begin{array}{lll} h = 152 \text{ mm} & b = 160 \text{ mm} & t_f = 9 \text{ mm} \\ t_w = 6 \text{ mm} & A = 38.8 \text{ cm}^2 & I_y = 1670 \text{ cm}^4 \\ I_z = 616 \text{ cm}^4 & i_y = 6.57 \text{ cm} & i_z = 3.98 \text{ cm} \\ I_w = 31400 \text{ cm}^6 & I_t = 12.3 \text{ cm}^4 & W_{el,y} = 220 \text{ cm}^3 \\ W_{el,z} = 76.9 \text{ cm}^3 & W_{pl,y} = 245 \text{ cm}^3 & W_{pl,z} = 118 \text{ cm}^3 \end{array} \quad (9.1)$$

Since the bending moment acts about y-y axis, the member might fail due to flexural buckling or due to lateral torsional buckling.

9.20.2 Cross-section classification

Tab. 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1$$

Flange:

$$c = \frac{160-6}{2} - 15 = 62 \text{ and}$$

$$\frac{c}{t} = \frac{62}{9} = 6.89 < 9\varepsilon = 9 \text{ (flange under compression)}$$

The flange belongs to class 1

Web:

The classification of the web is performed using the same procedure as in Example 9.2. In this case the web is examined under pure compressive force (unfavourable case).

$$\frac{d}{t} = \frac{152-2\cdot(9+15)}{6} = \frac{104}{6} = 17.33 < 33\varepsilon = 33$$

So, the web is class 1 and the whole section class 1. 5.5.2(6)

9.20.3 Verification to flexural and lateral torsional buckling

According to EN 1993-1-1, paragraph 6.3.3 (4), members that are subjected to combination of axial compressive force and bending moments, should satisfy the following equations:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leqslant 1$$ Eq. 6.61

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{y_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{y_{M1}}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{y_{M1}}} \leqslant 1$$ Eq. 6.62

In this case, since the section is class 1, and the moment $M_{z,Ed} = 0$, these equations become (with $\Delta M_{y,Ed} = \Delta M_{z,Ed} = 0$): Tab. 6.7

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} \leq 1, \qquad \frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y.Rk}}{\gamma_{M1}}} \leq 1$$

The factors k_{yy} and k_{zy} in this example will be calculated using Method 1 (Annex A, EN 1993-1-1).

9.20.3.1 Critical buckling lengths

$$l_{cr,y} = 1\cdot 500 = 500 \text{ cm} = l_{cr,z}$$

$$l_{cr,T} = 500\text{cm (simple forked support)}$$

9.20.3.2 Critical loads

It is:

$$N_{crit,y} = \frac{\pi^2 \mathrm{EI}_y}{l_{cr,y}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 1670}{500^2} = 1384.5\ \mathrm{kN}$$

$$N_{crit,z} = \frac{\pi^2 \mathrm{EI}_z}{l_{cr,z}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 616}{500^2} = 510.7\ \mathrm{kN}$$

The polar radius of gyration of the section in refer to shear center (which coincides here with the center of gravity), is:

$$i_M^2 = i_p^2 = i_y^2 + i_z^2 = 6.57^2 + 3.98^2 = 59\ \mathrm{cm}^2$$

and the elastic critical load for torsional buckling (equation 4.24) is:

$$N_{crit,T} = \frac{1}{i_M^2}\left(GI_t + \frac{\pi^2 \cdot EI_w}{l_T^2}\right) =$$

$$= \frac{1}{59}\left(\frac{2.1 \cdot 10^4}{2.6} \cdot 12.3 + \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 31400}{500^2}\right) = 2125\ \mathrm{kN}$$

9.20.3.3 Critical moment for lateral torsional buckling

The critical moment for lateral torsional buckling of a doubly symmetric section (z_j = 0), considering the influence of the end moments (C_2 = 0), and the simple forked support ($k = k_w$ = 1), is given from the relation:

$$M_{crit,LT} = C_1 \frac{\pi^2 EI_z}{l_{cr,T}^2} \cdot \sqrt{\frac{I_w}{I_z} + \frac{l_{cr,T}^2 \cdot G \cdot I_T}{\pi^2 \cdot E \cdot I_z}}$$ (Literature)

in which for $\psi = 0$:

$$C_1 = 1.88 - 1.40\psi + 0.52\psi^2 = 1.88 \leqslant 2.70$$

and finally

$$M_{crit,LT} = 1.88 \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 616}{500^2} \cdot \sqrt{\frac{31400}{616} + \frac{500^2 \cdot 12.3}{\pi^2 \cdot 2.6 \cdot 616}} = 15043\ \mathrm{kN\,cm}$$

9.20.3.4 Relative slenderness and reduction factors

For class 1 cross-section:

$$\bar{\lambda}_y = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}} = \sqrt{\frac{38.8 \cdot 23.5}{1384.5}} = 0.81$$ 6.3.1.2(1)

For $\frac{h}{b} = \frac{152}{160} = 0.95 < 1.2$ and $t_f = 9$ mm < 100 mm

The buckling curve is b, Tab. 6.2

and $\alpha = 0.34$ (imperfection factor) Tab. 6.1

$$\Phi_y = 0.5[1+\alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = \\ = 0.5[1+0.34(0.81-0.2)+0.81^2] = 0.932$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \overline{\lambda}_y^2}} = \frac{1}{0.932+\sqrt{0.932^2-0.81^2}} = 0.7179$$ Eq. 6.49

$$\bar{\lambda}_z = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}} = \sqrt{\frac{38.8 \cdot 23.5}{510.7}} = 1.336$$ 6.3.1.2(1)

For $\frac{h}{b} = 0.95 < 1.2$ and $t_f = 9$ mm < 100 mm.

The corresponding buckling curve for z-z axis is c and, $\alpha = 0.49$ (imperfection factor), so: $\Phi_z = 0.5[1+0.49(1.336-0.2)+1.336^2] = 1.671$

and

$$\chi_z = \frac{1}{1.671+\sqrt{1.671^2-1.336^2}} = 0.3738$$ 6.3.2.2(1)

Moreover:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl} \cdot f_y}{M_{cr}}} = \sqrt{\frac{245 \cdot 23.5}{15043}} = 0.619$$ Tab. 6.4

For a rolled section with $\frac{h}{b} = \frac{152}{160} = 0.95 < 2$ the buckling curve is a. Tab. 6.3

Imperfection factor $\alpha_{LT} = 0.21$

$$\Phi_{LT} = 0.5[1+\alpha_{LT}(\bar{\lambda}_{LT}-0.2)+\bar{\lambda}_{LT}^2] = 0.5[1+0.21(0.619-0.2)+0.619^2] = 0.735$$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = \frac{1}{0.735+\sqrt{0.735^2-0.619^2}} = 0.884 < 1$$ Eq. 6.56

In case that, alternatively, the above calculation is carried out according to EN 1993-1-1, paragraph 6.3.2.3, leads to: Tab. 6.5

For $\frac{h}{b} = 0.95 < 2$ the buckling curve is b

$$\alpha_{LT} = 0.34, \\ \Phi_{LT} = 0.5[1+\alpha_{LT}(\bar{\lambda}_{LT}-\bar{\lambda}_{LT,0})+\beta\bar{\lambda}_{LT}^2] = \\ = 0.5[1+0.34(0.619-0.4)+0.75 \cdot 0.619^2] = 0.681$$

and

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta \cdot \overline{\lambda}_{LT}^2}} = \frac{1}{0.681+\sqrt{0.681^2-0.75 \cdot 0.619^2}} = \\ = 0.908 < \min\left(1;\ \frac{1}{\bar{\lambda}_{LT}^2}\right) = \min\left(1;\ \frac{1}{0.619^2}\right) = 1$$ Eq. 6.57

9.20.3.5 Auxiliary terms of Method 1

Annex A, Tab. A.1

The calculations will be carried out using the elastic cross-section properties. 6.3.3(5)

It is:

$$\mu_y = \frac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-\chi_y\dfrac{N_{Ed}}{N_{cr,y}}} = \frac{1-\dfrac{200}{1384.5}}{1-0.7179\dfrac{200}{1384.5}} = 0.955$$

$$\mu_z = \frac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-\chi_z\dfrac{N_{Ed}}{N_{cr,z}}} = \frac{1-\dfrac{200}{510.7}}{1-0.3738\dfrac{200}{510.7}} = 0.713$$

$$a_{LT} = 1-\frac{I_T}{I_y} = 1-\frac{12.3}{1670} = 0.993$$

For $\psi_y = 0$:

$$C_{my,0} = 0.79+0.21\psi_i+0.36(\psi_i-0.33)\frac{N_{Ed}}{N_{cr,i}} =$$

Annex A, Tab. A.2

$$= 0.79+0.21\cdot 0+0.36(0-0.33)\frac{200}{1384.5} = 0.773.$$

For the calculation of nondimensional slenderness for lateral-torsional buckling due to uniform bending moment $\overline{\lambda}_0$, i.e. for ψ=1, (see previous paragraphs 9.20.3.3 and 9.20.3.4):

Annex A, Tab. A.1

$$C_1 = 1.88-1.40\psi+0.52\psi^2 = 1.88-1.40+0.52 = 1.0$$

$$M_{cr,LT} = \frac{1,0}{1,88}\cdot 15043 = 8002\ \text{kN cm and}$$

$$\bar{\lambda}_0 = \sqrt{\frac{220\cdot 23.5}{8002}} = 0.80$$

It is also:

Annex A, Tab. A.1

$$0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,TF}}\right)} =$$

$$= 0.2\sqrt{1.88}\sqrt[4]{\left(1-\frac{200}{510.7}\right)\left(1-\frac{200}{2125}\right)} = 0.236, \quad \text{so}$$

$$\bar{\lambda}_0 = 0.80 > 0.236 \quad \text{and :}$$

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\cdot\frac{A}{W_{el,y}} = \frac{10\cdot 100}{200}\cdot\frac{38,8}{22,0} = 0.88$$

$$C_{my} = C_{my,0} + (1 - C_{my,0}) \frac{\sqrt{\varepsilon_y} \cdot a_{LT}}{1 + \sqrt{\varepsilon_y} \cdot a_{LT}} =$$
$$= 0.773 + (1 - 0.773) \cdot \frac{\sqrt{0.88} \cdot 0.993}{1 + \sqrt{0.88} \cdot 0.993} = 0.882$$
$$C_{mLT} = C_{my}^2 \cdot \frac{a_{LT}}{\sqrt{\left(1 - \frac{N_{Ed}}{N_{crit,z}}\right)\left(1 - \frac{N_{Ed}}{N_{crit,T}}\right)}} =$$
$$= 0.882^2 \cdot \frac{0.993}{\sqrt{(1 - \frac{200}{510.7})(1 - \frac{200}{2125})}} = 1.041$$
$$k_{yy} = C_{my} \cdot C_{mLT} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,y}}} = 0.882 \cdot 1.041 \frac{0.955}{1 - \frac{200}{1384.5}} = 1.025$$
$$k_{zy} = C_{my} \cdot C_{mLT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,y}}} = 0.882 \cdot 1.041 \frac{0.713}{1 - \frac{200}{1384.5}} = 0.765$$

9.20.3.6 Verification

It is:

$$N_{Rk} = Af_y = 38.8 \cdot 23.5 = 912 \text{ kN}$$
$$M_{Rk} = W_{pl,y} \cdot f_y = 245 \cdot 23.5 = 5757 \text{ kNcm}$$

Tab. 6.7

and the interaction formulae are written:

$$\frac{200}{0.7179 \cdot \frac{912}{1.0}} + 1.025 \frac{1000}{0.92 \cdot \frac{5757}{1.0}} = 0.306 + 0.194 = 0.500 < 1$$
$$\frac{200}{0.3738 \cdot \frac{912}{1.0}} + 0.765 \frac{1000}{0.92 \cdot \frac{5757}{1.0}} = 0.587 + 0.144 = 0.731 < 1$$

Therefore, the member is sufficient.

Remark 12. If the column is laterally supported throughout its length, lateral torsional buckling is prevented, and the following apply:

$$\bar{\lambda}_0 = 0 \qquad \chi_{LT} = 1.0$$

Annex A, Tab. A.1

$$C_{my} = C_{my,0} = 0.773$$
$$C_{mz} = C_{mz,0} = 1.094$$
$$C_{mLT} = 1.0$$
$$k_{yy} = 0.773 \cdot 1 \cdot \frac{0.995}{1 - \frac{200}{1384.5}} = 0.863$$
$$k_{zy} = 0.773 \cdot 1 \cdot \frac{0.713}{1 - \frac{200}{1384.5}} = 0.644$$

and the interaction formulae are written:

$$\frac{200}{0.7179 \cdot \frac{912}{1.0}} + 0.863 \frac{1000}{1 \cdot \frac{5757}{1.0}} = 0.306 + 0.150 = 0.456 < 1$$

$$\frac{200}{0.3738 \cdot \frac{912}{1.0}} + 0.644 \frac{1000}{1 \cdot \frac{5757}{1.0}} = 0.586 + 0.112 = 0.698 < 1$$

9.20.4 Check of the resistance of sections at each end of the column

Upper end of the column: EN 1993-1-5, 6.3.3(2)

$$N_{Ed} = 200 \text{ kN}$$

$$M_{y,Ed} = 10 \text{ kNm}$$

$$V_{y,Ed} = \frac{M_{y,Ed}}{L} = \frac{10}{5} = 2 \text{ kN}$$

$$N_{Rd} = \frac{N_{Rk}}{\gamma_{M0}} = \frac{912}{1.0} = 912 \text{ kN}$$

$$M_{y,Rd} = \frac{M_{y,Rk}}{\gamma_{M0}} = \frac{57.57}{1.0} = 57.57 \text{ kNm}$$

$$V_{y,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{M0}} = \frac{13.24 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 179 \text{ kN} > 2 \text{ kN}$$

where

$$A_v = A - 2bt_f + (t_w + 2r)t_f = 38.8 - 2 \cdot 160.9 + (0.6 + 2 \cdot 1.5) \cdot 0.9 =$$
$$= 13.24 \text{ cm}^2 > \eta h_w t_w = 1.0 \cdot (15.2 - 2 \cdot 0.9) \cdot 0.6 = 8.04 \text{ cm}^2.$$

6.2.6(3)

So, it is taken $A_v = 13.24 \text{ cm}^2$.

Since: $V_{y,Ed} = 2 \text{ kN} < 0.5 V_{y,Rd} = 0.5 \cdot 179 = 89.5 \text{ kN}$,

the reduction due to shear force is not required.

As an approximation, for a cross-section of class 1. the following criterion might be used:

6.2.1(7), Eq. 6.2

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Ed}} \leq 1 \quad \text{or}$$

$$\frac{200}{912} + \frac{10}{57.57} + 0 = 0.22 + 0.17 = 0.39 < 1$$

Therefore, the section is sufficient.

9.21 Example: Beam under compression and bending, with intermediate lateral restraints

Verify the capacity of the simply supported IPE 600 beam shown in Fig. 9.31. with simple forked supports. The beam is laterally restrained at the points where the two concentrated loads apply. The design axial load is N_{Ed} = 880 kN and the design concentrated loads P_{Ed} = 240 kN. The loads P_{Ed} apply at the center of gravity of the section. Steel grade S 355.

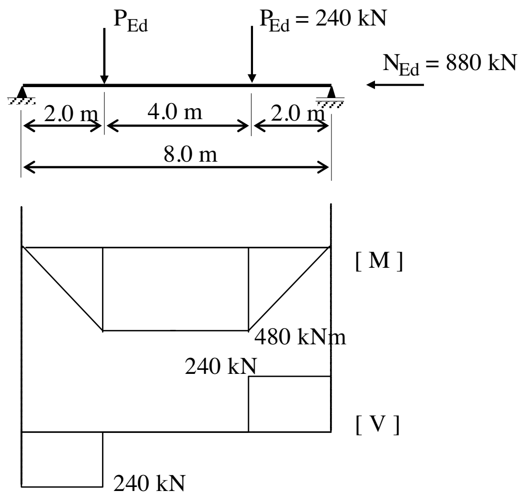


Fig. 9.31. Beam under compression and bending and diagrams with internal forces and moments

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.21.1 General

The verification will be performed using interaction formulas (6.61) and (6.62) from EN 1993-1-1 in combination with Method 1 (Annex A). In this example, these formulas are written as follows:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} \leqslant 1 \qquad \text{Eq. 6.61}$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} \leqslant 1 \qquad \text{Eq. 6.62}$$

9.21.2 Design resistance to axial compressive force

Classification of section IPE 600 for axial compressive force.

web:

$$\frac{c}{t} = \frac{514}{12} = 42.8 > 42\varepsilon = 42 \cdot 0.814 = 34.2 \qquad \text{Tab. 5.2. sheet 1}$$

so, the web belongs to class 4.

flange:

$$\frac{c}{t} = \left(\frac{220-12}{2} - 24\right)/19 = 4.2 < 9\varepsilon = 7.3 \qquad \text{Tab. 5.2. sheet 2}$$

the flange belongs to class 1 and the whole cross-section to class 4.

For the determination of the effective part of the web, it is (for $\psi = 1$):

$$\bar{\lambda}_\rho = (\bar{b}/t)/(28.4\varepsilon\sqrt{k_\sigma}) = (514/12)/28.4 \cdot 0.814 \cdot \sqrt{4} = 0.926$$

EN 1993-1-5. Eq. 4.3

$$\rho = \frac{\bar{\lambda}_\rho - 0.055(3+\psi)}{\bar{\lambda}_\rho^2} = \frac{0.926 - 0.055(3+1)}{0.9262} = 0.823$$

EN 1993-1-5. Eq. 4.2

so, the non-effective area of the web is:

$$\bar{A}_w = (1 - 0.823) \cdot 51.4 \cdot 1.2 = 10.92\ \text{cm}^2$$

and the non-effective length of the web is:

$$\overline{h}_w = (1 - 0.823) \cdot 51.4 = 9.10\ \text{cm}$$

Effective area of the cross-section

$$A_{eff} = A - \bar{A}_w = 156 - 10.92 = 145.08\ \text{cm}^2$$

It is obvious that the effective and the total area of the section have the same center of gravity and thus the term $\Delta M_{y,Ed}$ does not exist in formulas (6.61) and (6.62).

Second moment of area of the effective section

$$I_{y,eff} = 92080 - \frac{1}{12} \cdot 1.2 \cdot 9.10^3 = 92005\ \text{cm}^4$$

$$I_{z,eff} = I_z = 3390\ \text{cm}^4$$

Radiuses of gyration

$$i_{y,eff} = (I_{y,eff}/A_{eff})^{0.5} = (92005/145.08)^{0.5} = 25.2\ \text{cm}$$

$$i_z = 4.66\ \text{cm}$$

Slendernesses

$$\lambda_y = 800/25.2 = 31.7, \quad \overline{\lambda}_y = 31.7/76.4 = 0.42$$

Eq. 6.51

$$\lambda_z = 400/4.66 = 85.8, \quad \overline{\lambda}_z = 85.8/76.4 = 1.12$$

It was assumed (conservatively) that the central part of the beam with the biggest length will behave in case of out of plane buckling as a simply supported element.

***Reduction factors** x*

Since $h/b = 600 / 220 = 2.73 > 1.2$ and $t_f = 19 < 40$ mm, buckling curve a should be used for buckling about y-y axis, and buckling curves b for buckling about z-z axis.

Tab. 6.2

It is:

$$x_y = 0.9474 \quad x_z = 0.5234 \text{ and}$$

Tab. 6.7

$$N_{Rk} = A_{eff} f_y = 145.08 \cdot 35.5 = 5150\ \text{kN}$$

9.21.3 Calculation of reduction factor x_{LT} for lateral torsional buckling

It was assumed (conservatively) that the central part of the beam with 4.0 m length will behave in case of lateral torsional buckling as simply supported element with forked supports at its ends.

The critical moment for lateral torsional buckling should be then calculated from the relation (Literature):

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \left[\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z} \right]^{0.5}$$

in which

$C_1 = 1.0$ (member under uniform bending moment),
$L = 400$ cm
$J_t = 166\ \text{cm}^4$ (torsion constant)
$I_w = 2850000\ \text{cm}^6$ (warping constant), and
$G/E = 1/2.6.$

Finally: $M_{cr} = 148647$ kNcm and

$$\overline{\lambda}_{LT} = (W_{pl} \cdot f_y / M_{cr})^{0.5} = (3512 \cdot 35.5/148647)^{0.5} = 0.916,$$ 6.3.2.2

since h/b = 600/220 = 2.73 > 2, using buckling curve b, x_{LT} = 0.6510. Tab. 6.4

9.21.4 Resistance to bending of the cross-section

Classification of section: Tab. 5.2

Web:

$$\frac{c}{t} = 42.8 < 72\varepsilon = 72 \cdot 0.814 = 58.6 \quad \text{class 1}$$

Flange:

As previously class 1, thus the whole cross-section belongs to class 1.

Resistance to bending:

$$M_{y,Rk} = W_{pl} \cdot f_y = 3512 \cdot 35.5 = 124676\ \text{kNcm}$$ Tab. 6.7

9.21.5 Calculation of interaction factors k_{yy}, k_{zy}

9.21.5.1 Auxiliary terms (Table A1)

It is:

$$N_{cr,y} = \pi^2 EI_y / l_y^2 = \pi^2 \cdot 21000 \cdot 92080/800^2 = 29820\ \text{kN}$$

$$N_{Ed} = 880\,\text{kN}$$

$$\mu_y = \frac{1 - \dfrac{N_{Ed}}{N_{cr,y}}}{1 - x_y \dfrac{N_{Ed}}{N_{cr,y}}} = \frac{1 - \dfrac{880}{29820}}{1 - 0.9474 \dfrac{880}{29820}} = 0.998$$

Annex A, Tab. A.1

$$N_{cr,z} = \pi^2 EI_z / l_z^2 = \pi^2 \cdot 21000 \cdot 3390/400^2 = 4391\,\text{kN}$$

$$\mu_z = \frac{1 - \dfrac{N_{Ed}}{N_{cr,z}}}{1 - x_z \dfrac{N_{Ed}}{N_{cr,z}}} = \frac{1 - \dfrac{880}{4391}}{1 - 0.5234 \dfrac{880}{4391}} = 0.894$$

$$w_y = \frac{W_{pl,y}}{W_{el,y}} = \frac{3512}{3070} = 1.144, \qquad w_z = \frac{W_{pl,z}}{W_{el,z}} = 1.50$$

$$n_{p\lambda} = N_{Ed}/(N_{Rk}/\gamma_{M1}) = 880 \cdot 1.0/5150 = 0.171$$

$$\alpha_{LT} = 1 - \frac{I_t}{I_y} = 1 - \frac{166}{92080} = 0.998$$

9.21.5.2 Coefficient C_{my}

It is $\overline{\lambda}_o = \overline{\lambda}_{LT}$, and coefficient $C_{my,o}$ should be calculated for ψ = 1.0. It is:

$$C_{my,o} = 0.79 + 0.21\psi + 0.36(\psi - 0.33)\frac{N_{Ed}}{N_{cr,y}} =$$

Annex A, Tab. A.2

$$= 0.79 + 0.21 \cdot 1 + 0.36(1 - 0.33)\frac{880}{29820} = 1.007$$

$$\varepsilon_y = \frac{N_{y,Ed}}{N_{Ed}} \cdot \frac{A}{W_{el,y}} = \frac{48000}{880} \cdot \frac{156}{3070} = 2.772$$

Annex A, Tab. A.1

(class 1 cross-section)

$$C_{my} = C_{my,o} + (1 - C_{my,o})\frac{\sqrt{\varepsilon_y}\alpha_{LT}}{1 + \sqrt{\varepsilon_y}\alpha_{LT}} =$$

$$= 1.007 + (1 - 1.007)\frac{\sqrt{2.772} \cdot 0.998}{1.0 + \sqrt{2.772} \cdot 0.998} = 1.003$$

9.21.5.3 Coefficient C_{mLT}

Critical load for torsional buckling: Annex A, Tab. A.1

$$N_{cr,T} = \frac{1}{i_M^2}\left(GJ_t + \frac{\pi^2 EI_w}{l_T^2}\right)$$

(Literature)

In which for a doubly symmetric section it is:

$$i_M^2 = i_y^2 + i_z^2$$

and l_T buckling length for torsional buckling.

In this case: l_T = 4.0 m, so

$$i_M^2 = 24.3^2 + 4.66^2 = 612.21 \text{ cm}^2$$

$$N_{cr,T} = \frac{1}{612.21}\left(\frac{21000}{2.6}\cdot 166 + \frac{\pi^2 \cdot 21000 \cdot 2850000}{400^2}\right) = 8220 \text{ kN}$$

and

$$C_{mLT} = C_{my}^2 \frac{\alpha_{LT}}{\left[\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)\right]^{0.5}} =$$

$$= 1.003^2 \quad \frac{0.998}{\left[\left(1-\frac{880}{4391}\right)\left(1-\frac{880}{8220}\right)\right]^{0.5}} = 1.188$$

9.21.5.4 Coefficients C_{yy} and C_{zy}

Annex A, Tab. A.1

Based on the relations given in Table A.1 and since there is not any moment $M_{z,Ed}$, that creates bending about z-z axis, it results in b_{LT} = 0 and

$$C_{yy} = 1 + (w_y - 1)(2 - \frac{1,6}{W_y} C_{my}^2 \bar{\lambda}_{\max} - \frac{1,6}{W_y} C_{my}^2 \bar{\lambda}_{\max}^2) n_{p\lambda} =$$

$$= 1 + (1.144 - 1)(2 - \frac{1.6}{1.144}\cdot 1.003^2 \cdot 1.12 - \frac{1.6}{1.144}\cdot 1.003^2 \cdot 1.12^2)0.171 =$$

$$= 0.967 > W_{el,y}/W_{pl,y} = 3070/3512 = 0.874$$

In the same manner d_{LT} = 0 and

$$C_{zy} = 1 + (w_y - 1)\left(2 - 14\frac{C_{my}^2\,\bar{\lambda}_{\max}^2}{w_y^5}\right) n_{p\lambda} =$$

$$= 1 + (1.144 - 1)\left(2 - 14\frac{1.003^2 \cdot 1.12^2}{1.144^5}\right)\cdot 0.171 =$$

$$= 0.827 \geqslant 0.6\sqrt{\frac{w_y}{w_z}\frac{W_{el,y}}{W_{pl,y}}} = 0.6\sqrt{\frac{1.144}{1.5}\cdot\frac{3070}{3512}} = 0.458$$

9.21.5.5 Interaction factors k_{yy} and k_{zy}

$$k_{yy} = C_{my} C_{mLT} \frac{\mu_y}{1 - \dfrac{N_{Ed}}{N_{cr,y}}} \cdot \frac{1}{C_{yy}} =$$

$$= 1.003 \cdot 1.188 \cdot \frac{0.998}{1 - \frac{880}{29820}} \cdot \frac{1}{0.967} = 1.267$$

and

$$k_{zy} = C_{my} C_{mLT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,y}}} \cdot \frac{1}{C_{xy}} \cdot 0.6\sqrt{\frac{w_y}{w_z}} =$$

$$= 1.003 \cdot 1.188 \cdot \frac{0.894}{1 - \frac{880}{29820}} \cdot \frac{1}{0.827} \cdot 0.6\sqrt{\frac{1.144}{1.50}} = 0.695$$

9.21.6 Verification of the member

Formulae (6.61) and (6.62) referred in paragraph 9.21.1 are written as follows:

$$\frac{880 \cdot 1.0}{0.9474 \cdot 5150} + 1.267 \frac{48000 \cdot 1.0}{0.651 \cdot 124676} = 0.180 + 0.749 = 0.929 < 1.0$$

and

$$\frac{880 \cdot 1.0}{0.5234 \cdot 5150} + 0.695 \frac{48000 \cdot 1.0}{0.651 \cdot 124676} = 0.326 + 0.411 = 0.737 < 1.0$$

Therefore, the member is sufficient.

9.21.7 Verification in shear

It is: V_{Ed} = 240 kN Shear area 6.2.6(3)

$$A_v = 156 - 2 \cdot 22 \cdot 1.9 + (1.2 + 2 \cdot 2.4) \cdot 1.9 = 83.8\,\text{cm}^2 > \eta h_w t_w = 1.0 \cdot 60 \cdot 1.2 = 72\,\text{cm}^2$$

Plastic design shear resistance

$$V_{p\lambda \cdot Rd} = A_v (f_y/\sqrt{3})/\gamma_{M0} = 83.8(35.5/\sqrt{3})/1.0 = 1718\text{ kN} > V_{Ed}$$ Eq. 6.18

Since

$$V_{Ed} = 240\text{ kN} < 0.5 V_{p\lambda \cdot Rd} = 0.5 \cdot 1718 = 859\text{ kN}$$ 6.2.8(2)

the reduction of the bending resistance due to shear force could be neglected.

9.22 Example: Column with class 4 cross-section

Verify the adequacy of a column under compression. The height of the column is 6.5 m and the design compressive force is N_{Ed} = 3500 kN (Fig. 9.32a). The cross-section is shown in Fig. 9.32b. Steel quality S 235.

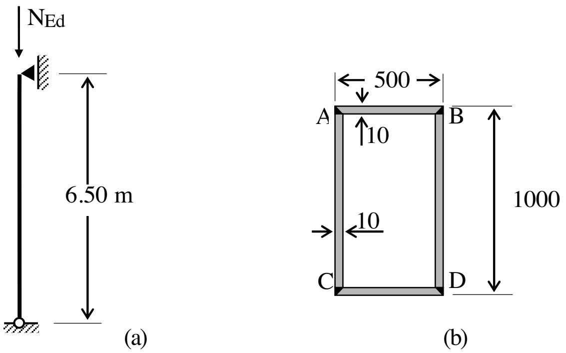


Fig. 9.32. Column with class 4 cross-section under compression

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.22.1 Cross-section classification (for uniform compression)

Elements AB, CD

$$c = 500 - 2 \cdot 10 = 480 \text{ mm}$$

$$\frac{c}{t} = \frac{480}{10} = 48 > 42\varepsilon = 42 \cdot 1 = 42$$ Tab. 5.2. sheet 1

These elements belong to class 4.

Elements AC, BD

$$c = 1000 - 2 \cdot 10 = 980 \text{ mm}$$

$$\frac{c}{t} = \frac{980}{10} = 98 > 42$$

These elements belong also to class 4.

Thus, the whole section is class 4.

9.22.2 Effective cross-section

Elements AB, CD

$$\psi = 1 \text{ (uniform compression)}, \ k_\sigma = 4$$ EN 1993-1-5. Tab. 4.1

$$\bar{b} = b - 2t = 50 - 2 \cdot 1 = 48 \text{ cm}$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28,4\varepsilon\sqrt{k_\sigma}} = \frac{48}{28.4 \cdot 1 \cdot \sqrt{4}} = 0.845$$ EN 1993-1-5. 4.4(2)

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{0.845 - 0.22}{0.845^2} = 0.875$$ EN 1993-1-5. Eq. 4.2

$$b_{eff} = \rho \cdot \bar{b} = 0.875 \cdot 480 = 420 \text{ mm}$$ EN 1993-1-5. Tab. 4.1

Elements AC, BD

$$\bar{b} = b_w = 980 \text{ mm for webs}$$ EN 1993-1-5. Tab. 4.1

$$\psi = 1 \text{ (uniform compression) } k_\sigma = 4$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{98}{28.4 \cdot 1 \cdot \sqrt{4}} = 1.725$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{1.725 - 0.22}{1.725^2} = 0.506$$ EN 1993-1-5. Eq. 4.2

$$b_{eff} = \rho \cdot \bar{b} = 0.506 \cdot 980 = 496 \text{ mm}$$ EN 1993-1-5. Tab. 4.1

The effective cross-section is shown in Fig. 9.33

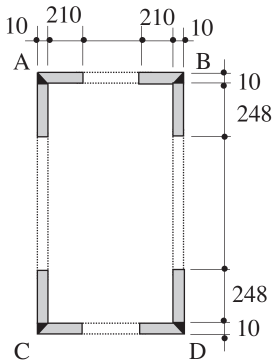


Fig. 9.33. Effective cross-section of the column

9.22.3 Cross-section verification

$$A_{eff} = 4 \cdot (24.8 + 1 + 21) \cdot 1 = 187.2\ \text{cm}^2$$

$$I_z = I_{\min} = 2 \cdot \frac{50^3}{12} \cdot 1 + 2 \cdot 98 \cdot 1 \cdot \left(\frac{49}{2}\right)^2 = 138482\ \text{cm}^4$$

$$N_{cr,z} = \frac{\pi^2 E I_z}{\ell^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 138482}{650^2} = 67933\ \text{kN}$$

$$\overline{\lambda}_z = \sqrt{\frac{A_{eff} f_y}{N_{cr,z}}} = \sqrt{\frac{187.2 \cdot 23.5}{67933}} = 0.25$$

Buckling curve *b*

$$\chi_z = 0.982$$

$$N_{b,Rd} = \chi_z A_{eff} f_y / \gamma_{M1} = 0.982 \cdot 187.2 \cdot 23.5 / 1.0 = 4320\ \text{kN}$$

Verification:

$$N_{Ed} = 3500\ \text{kN} < N_{b,Rd} = 4320\ \text{kN}.$$ Eq. 6.48

Due to the large torsional stiffness of the box section, lateral torsional buckling is not examined.

9.23 Example: Web of a plate girder under transverse concentrated load

The plate girder shown in Fig. 9.34a supports a column with a RHS 100x15 cross-section, that transfers through a base plate with 35 mm thickness a concentrated force P_{Ed} = 360 kN (Fig. 9.34b). In the position of this column the plate girder is also subjected to a bending moment M_{Ed} = 620 kNm and an axial compressive force N_{Ed} = 250 kN. Verify the local adequacy of the girder in the area under the column. The compression flange of the plate girder is laterally restrained. Steel grade S 235.

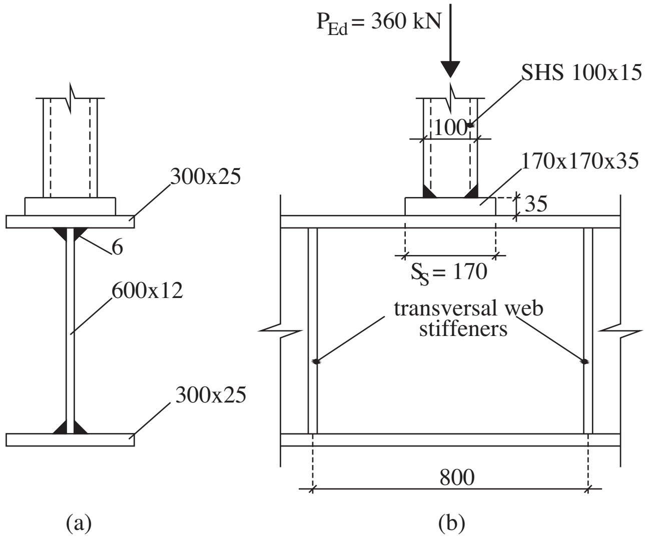


Fig. 9.34. Plate girder under transverse concentrated load

Note. In this design example, all the references in grey through the text refer to EN 1993-1-5, unless otherwise is written.

9.23.1 Resistance to transverse concentrated force

9.23.1.1 Effective loaded length l_y

6.5

It is:

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w} = \frac{23.5 \cdot 300}{23.5 \cdot 12} = 25 \quad \text{Eq. 6.8}$$

where $b_f = 300\text{ mm} < 2 \cdot (15\varepsilon t_f) = 2 \cdot 15 \cdot 1 \cdot 25 = 750\text{ mm}$

It is provisionally assumed (under confirmation) that:

$$\bar{\lambda}_F > 0.5 \text{ and thus:}$$

$$m_2 = 0.02\left(\frac{h_w}{t_f}\right)^2 = 0.02\left(\frac{60}{2.5}\right)^2 = 11.52 \quad \text{Eq.6.9}$$

and

$$l_y = s_s + 2t_f(1 + \sqrt{m_1 + m_2}) = 17 + 2 \cdot 2.5(1 + \sqrt{25.0 + 11.52}) = \\ = 52.2\text{ cm} < 80\text{ cm} \quad \text{Eq. 6.10}$$

(80 cm = distance between adjacent transverse stiffeners).

The length of stiff bearing s_s is considered as equal to the length of the column base plate, since, using a 45° distribution of stresses, it is derived that the entire length is effective. 6.3(1)

9.23.1.2 *Effective length for resistance to transverse concentrated forces* 6.4

Coefficient k_F

$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 = 6 + 2\left(\frac{60}{80}\right)^2 = 7.125$$ Fig. 6.1

Critical buckling load

$$F_{cr} = 0.90 k_F E \frac{t_w^3}{h_w} = 0.90 \cdot 7.125 \cdot 21000 \frac{1.2^3}{60} = 3878\ \text{kN}$$ Eq. 6.5

Relative slenderness

$$\overline{\lambda}_F = \sqrt{\frac{\lambda_y t_w f_{yw}}{F_{cr}}} = \sqrt{\frac{52.2 \cdot 1.2 \cdot 23.5}{3878}} = 0.62$$ Eq. 6.4

and $\overline{\lambda}_F = 0.62 > 0.50$. so the choice of eq. 6.9 for the calculation of m_2 was correct.

Reduction factor x_F

$$x_F = \frac{0.5}{\overline{\lambda}_F} = \frac{0.5}{0.62} = 0.806 < 1.0$$ Eq. 6.3

Effective length

$$L_{eff} = x_F \cdot l_y = 0.806 \cdot 52.2 = 42.0\ \text{cm}$$ Eq. 6.2

9.23.1.3 *Design resistance to transverse forces*

It is:

$$F_{Rd} = \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} = 23.5 \cdot 42.0 \cdot 1.2 = 1184\ \text{kN}$$ Eq. 6.1

Verification

$$n_2 = \frac{F_{Ed}}{F_{Rd}} = \frac{360}{1184} = 0.31 < 1.0$$ Eq. 6.14

9.23.1.4 *Verification to axial force and uniaxial bending moment*

Flange classification EN 1993-1-1. Tab. 5.2. sheet 2

$$\frac{c}{t} = \left(\frac{300 - 12}{2} - 6\sqrt{2}\right)/25 = 5.4 < 9\varepsilon = 9$$

Class 1.

Web classification EN 1993-1-1. Tab. 5.2. sheet 1

For bending:

$$c/t = (600 - 2 \cdot 6\sqrt{2})/12 = 48.6 < 72 \quad \text{class 1}$$

For axial compression

$$\frac{c}{t} = 48.6 > 42. \text{ class 4}$$

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{48.6}{28.4 \cdot 1.0\sqrt{4}} = 0.856 \qquad 4.4$$

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{0.856 - 0.055(3+1)}{0.856^2} = 0.868 \qquad \text{Eq. 4.2}$$

Effective area: $A_{eff} = 2 \cdot 30 \cdot 2.5 + 0.868 \cdot 60 \cdot 1.2 = 212.5 \text{ cm}^2$.

Second moment of area (class 1 section for bending)

$$I_y = \frac{1}{12} \cdot 1.2 \cdot 60^3 + 2 \cdot 30 \cdot 2.5 \cdot 31.25^2 = 168084 \text{ cm}^4$$

Elastic section modulus

$$W_{el,y} = I_y/y_{\max} = 168084/32.5 = 5172 \text{ cm}^3$$

Verification 4.6

$$n_1 = \frac{N_{Ed}}{\frac{f_y A_{eff}}{\gamma_{M0}}} + \frac{M_{Ed}}{\frac{f_y W_{eff}}{\gamma_{M0}}} = \frac{250 \cdot 1.0}{23.5 \cdot 212.5} + \frac{62000 \cdot 1.0}{23.5 \cdot 5172} = 0.05 + 0.51 = 0.56 < 1$$

Eq. 4.14

9.23.2 Interaction between transverse force, bending moment and axial force

7.2

$$n_2 + 0.8n_1 = 0.31 + 0.8 \cdot 0.56 = 0.76 < 1.4 \qquad \text{Eq. 7.2}$$

9.23.3 Flange induced buckling

8

To prevent the compression flange buckling in the plane of the web, the following criterion should be met:

$$\frac{h_w}{t_w} \leqslant k\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}} \qquad \text{Eq. 8.1}$$

where

$$A_w = 60 \cdot 1.2 = 72 \text{ cm}^2$$

$$A_{fc} = 30 \cdot 2.5 = 75 \text{ cm}^2$$

$k = 0.4$ (it is assumed that plastic moment resistance is utilized for the determination of the beam's resistance)

and

$$\frac{h_w}{t_w} = \frac{60}{1.2} = 50 \leqslant 0.40\frac{21000}{23.5}\sqrt{\frac{72}{75}} = 350$$

9.24 Example: Laced built-up column

The built-up columns AB and CD of the steel frame shown in Fig. 9.35a, have a cross-section shown in Fig. 9.35b and are fixed in both directions at their base. The verification of their capacity is required for design loads P_{Ed} = 1890 kN and for steel grade S 235. The out of plane movement of the joints B, D is prevented. The equivalent buckling length coefficient of the columns for in plane buckling of the columns is approximately calculated equal to 1.15.

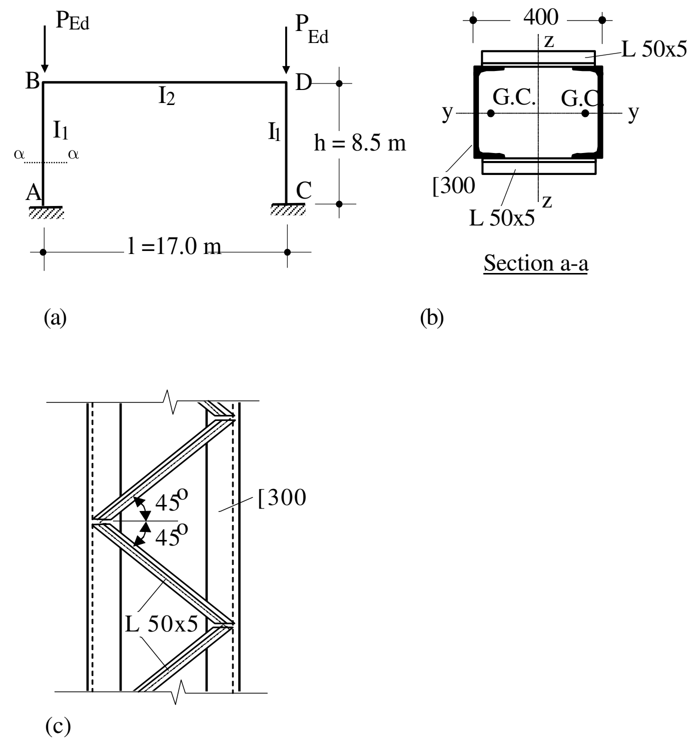


Fig. 9.35. Built-up columns in a portal frame

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.24.1 Buckling of built-up columns about y-y axis (out of the frame's plane)

Slenderness

$$\lambda_y = 0.70 \cdot 850/11.7 = 50.9$$

$$\overline{\lambda}_y = \lambda_y/\lambda_1 = 50.9/93.9 = 0.542$$

Design resistance and verification

Buckling curve c, $\chi_y = 0.819$ Tab. 6.2

$$N_{b,Rd} = 0.819 \cdot 2 \cdot 58.8 \cdot 23.5/1.0 = 2263 \text{ kN} > N_{Ed} = 1890 \text{ kN}$$ Eq. 6.47

The cross-section (under compression) belongs to class 1. since: Tab. 5.2

Web:

$$\frac{c}{t} = \frac{232}{10} = 23.2 < 33\varepsilon = 33 \text{ and}$$

Flange: $c/t = (100-10-16)/16 = 4.63 < 9$

9.24.2 Buckling of built-up columns about z-z axis

Effective second moment of area of laced built-up column

$$I_{eff} = 0.5h_0^2 A_{ch} = 0.5 \cdot (40 - 2 \cdot 2.70)^2 \cdot 58.8 = 35197 \text{ cm}^4$$ Eq. 6.72

The critical buckling load of the built-up column is derived from the following formula:

$$N_{cr} = \pi^2 EI_{eff}/(kl)^2 = \pi^2 \cdot 21000 \cdot 35197/(1.15 \cdot 850)^2 = 7635 \text{ kN}$$ 6.4.1(6)

Shear stiffness of lacings Fig. 6.9

$$S_v = nEA_d \alpha h_0^2/(2d^3) = 2 \cdot 21000 \cdot 4.8 \cdot 34.6^2 \cdot 69.2/(2 \cdot 48.93^3) = 71284 \text{ kN}$$

Initial imperfection

$$e_0 = kl/500 = 1.15 \cdot 850/500 = 1.955 \text{ cm}$$

Second order bending moment

$$M_{Ed} = \frac{N_{Ed} e_0}{\left(1 - \frac{N_{Ed}}{N_{cr}} - \frac{N_{Ed}}{S_V}\right)} = \frac{1890 \cdot 1.955}{\left(1 - \frac{1890}{7635} - \frac{1890}{71284}\right)} = 5090 \text{ kNcm}$$

(Since no transversal loading is applied to the beam BD, and its self weight is neglected in this example, no moments arise at the columns).

Design chord axial force

$$N_{ch.Ed} = 0.50 N_{Ed} + \frac{M_{Ed} h_0 A_{ch}}{2Ieff} = 0.50 \cdot 1890 + \frac{5090 \times 34,6 \times 58,8}{2 \times 35197} = 1092 \text{ kN}$$

Eq. 6.69

Slenderness

$$\lambda = 69.2/2.90 = 23.9$$ Fig. 6.8

(main member between two adjacent joints)

$$\overline{\lambda} = 23.9/93.9 = 0.254$$

Design resistance-Verification

Buckling curve c, $\chi = 0.972$

$$N_{b,Rd} = 0.972 \cdot 58.8 \cdot 23.5/1.0 = 1343 \text{ kN} > N_{ch.Ed} = 1092 \text{ kN}$$ Eq. 6.47

9.24.3 Verification of the lacings

6.4.1(7)

Shear force

$$V_{Ed} = \pi M_{Ed}/(kl) = \pi \cdot 5090/(1.15 \cdot 850) = 16.4 \text{ kN}$$ Eq. 6.70

Axial compressive force per diagonal lacing

$$N_{Ed} = V_{Ed} d/(nh_0) = 16.4 \cdot 48.93/(2 \cdot 34.6) = 11.6 \text{ kN}$$

Slenderness

$$\lambda = 48.93/0.98 = 50$$

$$\bar{\lambda} = 50/93.9 = 0.53$$

Design resistance-verification

Buckling curve b, $\chi = 0.871$ Tab. 6.2

$$N_{b.Rd} = 0.871 \cdot 4.80 \cdot 23.5/1.0 = 98.2 \text{ kN} > N_{Ed} = 11.6 \text{ kN}$$

9.25 Example: Built-up column under axial force and bending moment

Verify the capacity of the cantilever built-up column with lacing bars shown in Fig. 9.36. The design vertical and horizontal loads are: N_{Ed} = 300 kN and H_{Ed} = 45 kN. Steel grade S 235.

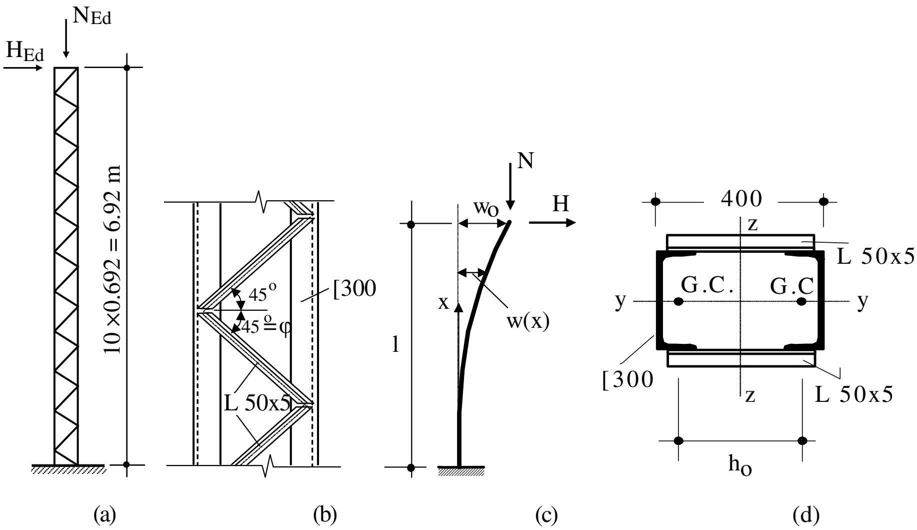


Fig. 9.36. Built-up column (cantilever) with horizontal and vertical forces at the top

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.25.1 Internal forces and moments

The initial imperfection of a cantilever (see Fig. 9.36c) may be written in the form of a sinusoidal function, as:

$$w = w_0\left(1 - \cos\frac{\pi x}{2l}\right)$$

(w_0 is the initial imperfection at the end of the cantilever).

When such a cantilever is subjected to vertical and horizontal loads, N and H (Fig. 9.36c), the elastic critical buckling load as well as the variation of bending moments and shear forces along the cantilever can be expressed as:

a) Elastic critical buckling load:

$$N_{cr} = \frac{1}{\frac{4\ell^2}{\pi^2 EI^*} + \frac{1}{S_V}}$$

in which I^* is the second moment of area of the built-up column and S_V its shear stiffness.

b) Bending moment of the built-up column at any point through its x axis:

$$M(x) = H(l-x) + \frac{N}{1-\frac{N}{N_{cr}}} w_0 \cos\frac{\pi x}{2l}$$

c) Shear force

$$V(x) = H + \frac{N}{1-\frac{N}{N_{cr}}}\frac{w_0}{2l}\pi\sin\frac{\pi x}{2l}$$

9.25.2 Maximum axial force at the unfavorable chord

Equivalent buckling length coefficient $k = 2$.

Initial imperfection:

$$e_0 = 2l/500 = 2\cdot 692/500 = 2.8 \text{ cm}$$ 6.4.1(1)

Effective second moment of area of the built-up column

$$I_{eff} = 0.5h_0^2 A_{ch} = 0.5\cdot(40-2\cdot 2.70)^2\cdot 58.8 = 35197 \text{ cm}^4$$ 6.4.2.1(4)

Shear stiffness

$$S_v = 2EA_d a h_0^2/(2d^3) = 2\cdot 21000\cdot 4.8\cdot 34.6^2\cdot 69.2/(2\cdot 48.93^3) = \\ = 71284 \text{ kN}$$ Fig.6.9

and

$$N_{cr} = \frac{1}{\frac{4\cdot 692^2}{\pi^2\cdot 21000\cdot 35197} + \frac{1}{71284}} = 3615 \text{ kN}$$

The maximum bending moment is at the bottom of the cantilever. The general formula of the previous paragraph, for $x = 0$ leads to:

$$\max M_{Ed} = H_{Ed} l + \frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} e_0 = 45 \cdot 692 + \frac{300}{1 - \frac{300}{3615}} \cdot 2.8 = 32056 \text{ kNcm}$$

Axial force at the most loaded chord: Eq. 6.69

$$N_{ch.Ed} = 0.50 N_{Ed} + \frac{M_{Ed} h_0 A_{ch}}{2 I_{eff}} = 0.50 \cdot 300 + \frac{32056 \cdot 34.6 \cdot 58.8}{2 \cdot 35197} = 1076 \text{ kN}$$

9.25.3 Buckling verification of each chord

$$\lambda = 69.2/2.90 = 23.9$$

$$\overline{\lambda} = \lambda/\lambda_1 = 23.9/93.9 = 0.254$$

Buckling curve c, $\chi = 0.972$ Tab. 6.2

$$N_{b.Rd} = 0.972 \cdot 58.8 \cdot 23.5/1.0 = 1343 \text{ kN} > N_{ch.Ed} = 1076 \text{ kN}$$ Eq. 6.47

9.25.4 Verification of the diagonal lacings

6.4.1(7)

The maximum shear force develops at the top of the cantilever. By applying the general formula of the first paragraph for $x = l$ it is obtained:

$$\max V_{Ed} = H_{Ed} + \frac{N_{Ed}}{1 - \frac{N_{Ed}}{N_{cr}}} w_0 \frac{\pi}{2l} = 45 + \frac{300}{1 - \frac{300}{3615}} \cdot 2.8 \cdot \frac{\pi}{2 \cdot 692} = 47.1 \text{ kN}$$

Design compression force per diagonal lacing:

$$N_{Ed} = \max V_{Ed} \,/(2\cos\phi) = 47.1/(2 \cdot 0.707) = 33.3 \text{ kN}$$

$$\lambda = 48.93/0.98 = 50$$

$$\overline{\lambda} = 50/93.9 = 0.53$$

Buckling curve b, $\chi = 0.871$ Tab. 6.2

$$N_{b.Rd} = 0.871 \cdot 4.8 \cdot 23.5/1.0 = 98.2 \text{ kN} > N_{Ed} = 33.3 \text{ kN}$$ Eq. 6.47

9.26 Example: Built-up column with battens

Verify the capacity of the built-up battened column AB shown in Fig. 9.37a, with a cross-section as in Fig. 9.37b. The beam BC is considered very stiff in comparison to the column. The applied vertical design load at the top of the column is P_{Ed} = 2300 kN. The joint B and the mid span of the column AB are laterally restrained in the out of plane direction. Steel grade S 355.

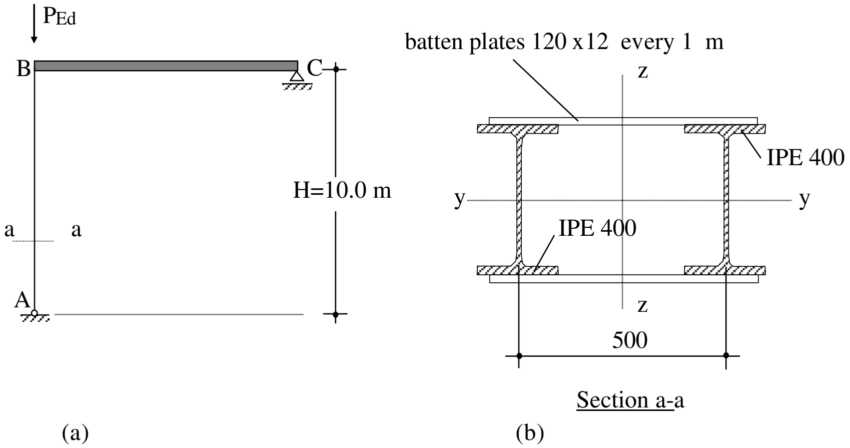


Fig. 9.37. Frame with a built-up column

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.26.1 Overall buckling about y-y axis

The IPE 400 cross-section of the built-up column is classified as class 4 (compression element), since:

Flange:

$$\frac{c}{t} = \left[\frac{180-8.6}{2} - 21\right]/13.5 = 4.8 < 9\varepsilon = 7.3.$$

class 1

Web:

$$\frac{c}{t} = 331/8.6 = 38.5 > 42\varepsilon = 34.2 \quad \text{class 4.}$$

Determination of the effective area of the web:

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{\kappa_\sigma}} = \frac{38.5}{28.4\,0.814 \cdot \sqrt{4.0}} = 0.832$$

EN 1993-1-5. 4.4(2)

Where $\kappa_\sigma = 4$ from Tab. 4.1 for $\psi = 1$.

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} = \frac{0.832 - 0.055(3+1)}{0.832^2} = 0.88 < 1.0$$

EN 1993-1-5. Eq. 4.2

$$b_{eff} = \rho\bar{b} = 0.88 \cdot 331 = 291 \text{ mm}$$

EN 1993-1-5. Tab. 4.1

Non-effective part of the web: $b_{ineff} = 331 - 291 = 40$ mm

Properties of the effective section

$$I_{y,eff} = 23130 - \frac{1}{12} \cdot 0.86 \cdot 4^3 = 23125\ \text{cm}^4$$
$$A_{ch,eff} = 84.5 - 4 \cdot 0.86 = 81.1\ \text{cm}^2$$
$$i_{y,eff} = (23125/81.1)^{0.5} = 16.9\ \text{cm}, \text{ and therefore:}$$

Slenderness

$$\lambda_y = 0.5H/i_{y,eff} = 500/16.9 = 29.6$$

Relative slenderness

$$\overline{\lambda} = \lambda_y/\lambda_1 = \frac{29.6}{93.9\varepsilon} = \frac{29.6}{93.9 \cdot (235/355)^{0.5}} = 0.387$$ 6.3.1.3

Buckling curve *a*,

$$\chi_y = 0.957, \text{ and:}$$ Tab. 6.2
$$N_{b.Rd} = \chi_y 2A_{ch,eff} f_y/\gamma_{M1} = 0.957 \cdot 2 \cdot 81.1 \cdot 35.5/1.0 =$$
$$= 5510.5\ \text{kN} > P_{Ed} = 2300\ \text{kN}$$ Eq. 6.47

9.26.1.1 Verification of the built-up column 6.4

a) Initial imperfection
Equivalent buckling length factor $k = 2$ (the rotation of the column head is considered as negligible due to the big stiffness of the beam BC)

$$e_0 = 2 \cdot 1000/500 = 4.0cm$$ 6.4.1(1)

b) Effective second moment of area of the built-up section.
The second moment of area is initially calculated for $\mu = 1$

$$I_{eff} = 0.5h_0^2 A_{ch} + 2\mu I_{ch} = 0.5 \cdot 50^2 \cdot 83.0 + 2 \cdot 1320 = 106390\ \text{cm}^4$$ Eq. 6.74

(the second moment of area of the effective section about z-z axis is considered here as practically equal to the second moment of area of the gross section).
The corresponding radius of gyration is

$$i_o = (I_1/2 \cdot A_{ch})^{0.5} = (106390/2 \cdot 83.0)^{0.5} = 25.3\ \text{cm}$$

and the slenderness is

$$\lambda = 2 \cdot 1000/25.3 = 79$$

Since $75 < \lambda = 79 < 150$, Tab. 6.8
$\mu = 2 - \frac{\lambda}{75} = 2 - \frac{79}{75} = 0.947$. and therefore

$$I_{eff} = 0.5h_0^2 A_{ch} + 2\mu I_{ch} = 0.5 \cdot 50^2 \cdot 83.0 + 2 \cdot 0.947 \cdot 1320 =$$
$$= 106250\ \text{cm}^4$$ Eq. 6.74

c) Shear stiffness of the column
In-plane second moment of area of each batten

$$I_b = \frac{1}{12} \cdot 1.2 \cdot 12^3 = 172.8 \text{ cm}^4$$

$$S_V = \frac{24EI_{ch}}{a^2\left[1+\frac{2I_{ch}}{nI_b}\frac{h_0}{a}\right]} = \frac{24 \cdot 21000 \cdot 1320}{100^2 \cdot \left[1+\frac{2 \cdot 1320}{2 \cdot 172.8} \cdot \frac{50}{100}\right]} = 13804 \text{ kN} <$$

$$< \frac{2\pi^2 EI_{ch}}{a^2} = \frac{2 \cdot \pi^2 \cdot 21000 \cdot 1320}{100^2} = \quad 54717 \text{ kN}$$ Eq. 6.73

$n = 2$ is the number of the planes of battens Fig. 6.9

d) Critical buckling load of the column 6.4.1(6)

$$N_{cr} = \pi^2 EI_{eff}/(kl)^2 = \pi^2 \cdot 21000 \cdot 106250/(2 \cdot 1000)^2 = 5505 \text{ kN}$$

e) Design value of the maximum moment in the middle of the column considering second order effects 6.4.1(6)

$$M_{Ed} = \frac{P_{Ed}e_0}{1-\frac{P_{Ed}}{N_{cr}}-\frac{P_{Ed}}{S_V}} = \frac{2300 \cdot 4}{1-\frac{2300}{5505}-\frac{2300}{13804}} = 22138 \text{ kNcm}$$

f) Internal forces in each chord of the column
Axial force Eq. 6.69

$$N_{ch,Ed} = 0.50(P_{Ed} + M_{Ed}h_0A_{ch}/I_{eff}) =$$
$$= 0.50 \cdot (2300 + 22138 \cdot 50 \cdot 83.0/106250) = 1582 \text{ kN}$$

Shear force Eq. 6.70

$$V_{Ed} = \pi M_{Ed}/(kl) = \pi \cdot 22138/(2 \cdot 1000) = 34.77 \text{ kN}$$

and for each chord:

$$V'_{Ed} = \frac{1}{2} \cdot 34.77 = 17.39 \text{ kN}$$

g) Verification of adequacy 6.4.3
Each chord should be checked to compression and bending with the following data: an axial force $N_{ch,Ed}$, a moment $M_{ch,Ed} = 17.39 \cdot 50 = 869.5$ kN cm, length equal to the half distance between two pairs of adjacent battens, equivalent buckling length factor $k = 2.0$ related to buckling about its minor principal axis. 6.4.3.1

The verification should be carried out using Eq. 6.62 (buckling about the minor principal axis), which leads to the following relation: 6.3.3

$$\frac{N_{ch,Ed}}{\frac{x_z A_{eff} f_y}{\gamma_{M1}}} + k_{zz}\frac{M_{ch,Ed}}{\frac{W_{el,z} f_y}{\gamma_{M1}}} \leq 1$$

The bending resistance should be calculated elastically, using Method 1 of Annex A.

Slenderness of each chord Fig. 6.11

$$\lambda_z = 2 \cdot 50/3.95 = 25.3 \text{ and}$$
$$\bar{\lambda}_z = 25.3/(93.9\varepsilon) = 25.3/(93.9 \cdot 0.814) = 0.33.$$

For buckling curve b it is x_z = 0.953 Tab. 6.2

Auxiliary terms and factors Annex A, Tab. A.1

$$N_{cr,z} = \pi^2 EI_z/(kl)^2 = \pi^2 \cdot 21000 \cdot 1320/(2 \cdot 50)^2 = 27358 \text{ kN}$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - x\frac{N_{Ed}}{N_{cr,z}}} = \frac{1 - \frac{1582}{27358}}{1 - 0.953\frac{1582}{27358}} = 0.997$$

$$\psi_z = 0.79 \cdot C_{mz} = 0.79 \cdot 0.36 \cdot 0.33 \cdot \frac{1582}{27358} = 0.783$$ Annex A, Tab. A.2

$$k_{zz} = C_{mz}\frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,z}}} = 0.783 \cdot \frac{0.997}{1 - \frac{1582}{27358}} = 0.829$$

and therefore:

$$\frac{1582}{0.953 \cdot 83.0 \cdot 35.5} + 0.829\frac{869.5}{146 \cdot 35.5} = 0.702 < 1$$

that is the section is adequate.

9.26.2 Verification of battens

Shear force per batten Fig. 6.11

$$T_{Ed} = V_{Ed}a/2h_o = 34.77 \cdot 100/(2 \cdot 50) = 34.77 \text{ kN}$$

Bending moment

$$M_{b,Ed} = 34.77 \cdot \frac{1}{2} \cdot 50 = 869.3 \text{ kNcm}$$

Verification for bending resistance

$$M_{b,Ed} = 869.3 < M_{Rd} = \frac{1}{6} \cdot 1.2 \cdot 12^2 \cdot \frac{35.5}{1.0} = 1022.4 \text{ kNcm}$$

Verification for shear

$$\tau_{Ed} = \frac{3}{2}\frac{34.77}{1.2 \cdot 1.2} = 3.62 \text{ kN/cm}^2 < \tau_{Rd} = \frac{f_y}{\sqrt{3} \cdot \gamma_{M0}} = \frac{35.5}{\sqrt{3} \cdot 1} = 20.5 \text{ kN/cm}^2$$

Remark 13. In case where the length of the battens is relatively large, the verification of their adequacy should be done considering their resistance to lateral torsional buckling according to bending moments' diagram shown in Fig. 6.11 of EN 1993-1-1.

The welding between battens and chords (it is not included in this example) could be verified according to Example 9.45.

9.27 Example: Closely spaced built-up members under compression

A built-up compressive bar of a truss has a length of 3.20 m and consists of two closely spaced angle members 80 × 8. The distance between the angles is 15 mm and they are connected every 228.5 mm through 60 × 50 × 15 packing plates. The steel grade is S 235. The determination of the design resistance of this bar is required for the following cases:

1. Arrangement of angles as in Fig. 9.38a.
2. Arrangement of star-battened angles as in Fig. 9.38b.

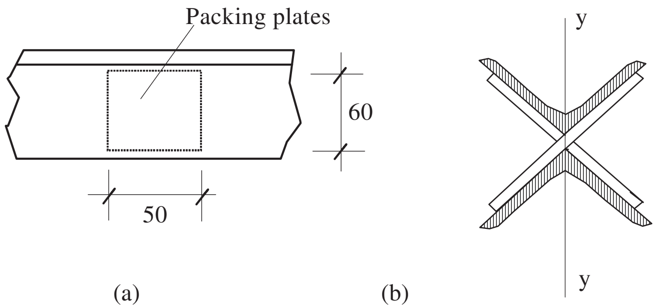


Fig. 9.38. Closely spaced built-up members

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.27.1 Arrangement of angles back-to-back

6.4.4

Since

$$22.85 < 15i_{\min} = 15 \cdot 1.55 = 23.25 \text{ cm}$$ Tab. 6.9

(where $i_{\min}$ is the minimum radius of gyration of each angle), the bar should be checked as a single integral member ignoring the effect of shear stiffness. Therefore:

$$\lambda_x = 320/2.42 = 132.2, \quad \lambda_1 = 93.9\,\varepsilon = 93.9$$
$$\overline{\lambda}_x = 132.2/93.9 = 1.41$$

For buckling curve b, $\chi = 0.377$ Tab. 6.2

Design resistance of the bar

$$N_{b,Rd} = 0.377 \cdot 2 \cdot 12.3 \cdot 23.5/1.0 = 217.9 \text{ kN}$$ Eq.6.47

(since $80/8 = 10 < 15\varepsilon = 15$. the cross-section belongs to class 3). Tab. 5.2. sheet 3

9.27.2 Arrangement of star-battened angles as in Fig. 9.38b

Pairs of battens are placed according to Fig. 9.38b, subdividing the bar in three equal parts, i.e. every 320 / 3 = 106.7 cm. Fig. 6.13. Tab. 6.9

Since

$$106.7 \text{ cm} < 70 i_{\min} = 70 \cdot 1.55 = 108.5 \text{ cm}$$

the bar should be checked as a single integral member for buckling about minor y-y axis (Fig. 9.38b).

$$\lambda = 320/3.06 = 104.6, \quad \overline{\lambda} = 104.6/93.9 = 1.114$$

From buckling curve *c*, $\chi = 0.478$

and the design resistance of the bar is:

$$N_{b,Rd} = 0.478 \cdot 2 \cdot 12.3 \cdot 23.5/1.0 = 276.3 \text{ kN}$$ Eq. 6.47

Remark 14. If the distance between the packing plates is larger than the above, the bar should be checked as a built-up member, considering the influence of its shear stiffness.

9.28 Example: Joint and bars' verification in a truss with circular hollow sections (CHS)

A simply supported truss shown in Fig. 9.39a consists of circular hollow section (CHS) bars. The brace bars are directly connected (i.e. without the use of gusset plates) to the chords by welding. The verification of joint 2 as well as of the bars connected to this joint is required. Design load $P_{Ed} = 20$ kN. Steel grade S 235.

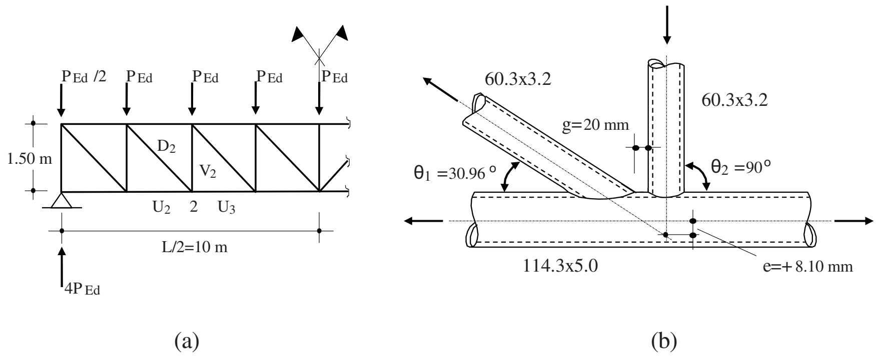


Fig. 9.39. Geometrical properties of the truss (a) and detail of joint 2 (b)

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.28.1 Verification of bars

9.28.1.1 The analysis of the truss leads to the following axial forces applied to the bars that are connected to joint 2:

$$U_2 = 116.67 \text{ kN (tension)}$$
$$U_3 = 200.00 \text{ kN (tension)}$$
$$D_2 = 97.18 \text{ kN (tension)}$$
$$V_2 = 50.00 \text{ kN (compression)}$$

The joints of the truss are considered as pinned, since the conditions of EN 1993-1-8. paragraphs 5.1.5 (3) + 7.4.1 are satisfied i.e.:

- the geometry of the joint is within the range of validity given in Table 7.1 (see next paragraph 9.28.2.1 below) and:
- $\frac{l_o}{d_o} = \frac{250}{11.43} = 21.9 > 6$
- $\frac{h_{V2}}{d_2} = \frac{150}{6.03} = 24.9 > 6$

9.28.1.2 Cross-section classification

7.1.2(2)

Bar 60.3 × 3.2

$$\frac{d}{t} = \frac{60.3}{3.2} = 18.84 < 50\varepsilon^2 = 50\left(\sqrt{\frac{235}{235}}\right)^2 = 50$$ EN 1993-1-1. Tab. 5.2. sheet 3

Therefore, the cross-section belongs to class 1.

9.28.1.3 Verification Bar V_2 60.3 × 3.2 (under compression)

$$\lambda = \frac{0.75 \cdot 150}{2.02} = 55.69$$

The equivalent buckling length factor for trusses with parallel chords can be taken equal to 0.75. EN 1993-1-1. Annex BB, 1.3.3

$$\lambda_1 = 93.9\varepsilon = 93.9$$
$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{55.69}{93.9} = 0.593$$ EN 1993-1-1. Eq. 6.50

Buckling curve, a EN 1993-1-1. Tab. 6.2

Imperfection factor $\alpha = 0.21$ EN 1993-1-1. Tab. 6.1

$$\Phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] = 0.5[1 + 0.21(0.593 - 0.2) + 0.593^2] = 0.717$$

and

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0.717 + \sqrt{0.717^2 - 0.593^2}} = 0.892$$

Area of cross-section

$$A = 5.74\ \text{cm}^2$$

$$N_{Rd} = 0.892 \cdot 5.74 \cdot 23.5/1.0 = 120.3\ \text{kN} > 50\ \text{kN}$$ 7.2.1(1)

Bar D_2 60.3 × 3.2 (under tension)

$$N_{Rd} = 5.74 \cdot 23.5/1.0 = 134.9\ \text{kN} > 97.18\ \text{kN}$$

Bars $U_{2.}$ U_3 114.3 × 5 (under tension)

Area of cross-section A = 17.2 cm^2

$$N_{Rd} = 17.2 \cdot 23.5/1.0 = 404.2\ \text{kN} > 200\ \text{kN}$$

9.28.2 Verification of joint 2

The joint 2 is a gap type welded N shaped joint and the bars are circular hollow sections. The verification should be performed according to Table 7.2. provided that the conditions of Table 7.1 are satisfied.

9.28.2.1 Range of validity according to Table 7.1

The following are valid:

$$0.2 \leqslant \frac{d_i}{d_o} = \frac{60.3}{114.3} = 0.53 \leqslant 1.0$$

$$10 \leqslant \frac{d_o}{t_o} = \frac{114.3}{5} = 22.86 \leqslant 50 \text{ (class 1)}$$

$$10 \leqslant \frac{d_i}{t_i} = \frac{60.3}{3.2} = 18.84 \leqslant 50 \quad \text{and}$$

$g = 20$ mm $\geqslant t_1 + t_2 = 2 \cdot 3.2 = 6.4$ mm. Thus Table 7.2 of EN 1993-1-8, paragraph 7.4.2 can be applied.

9.28.2.2 Field of application verification

Cross-sections belong to class 1 7.1.2(2)

$$f_y = 235\ \text{N/mm}^2 < 460\ \text{N/mm}^2$$ 7.1.1(4)

$$t_i = 3.2\ \text{mm} > 2.5\ \text{mm}$$ 7.1.1(5)

$$t_o = 5\ \text{mm} < 25\ \text{mm}$$ 7.1.1(6)

$$\min \theta_i = 30.96^\circ > 30^\circ$$ 7.1.2(3)

$$e = 8.10\ \text{mm} < 0.25 d_o = 0.25 \cdot 114.3 = 28.6\ \text{mm}$$ 5.1.5(5)

(so, moments resulting from eccentricities may be neglected).

9.28.2.3 Design resistance according to Table 7.2

a) Chord face failure

It is:

$$\gamma = \frac{d_o}{2t_o} = \frac{114.3}{2 \cdot 5} = 11.43 \qquad 1.4(6)$$

$$\frac{g}{t_o} = \frac{20}{5} = 4$$

$$k_g = \gamma^{0.2}\left[1 + \frac{0.024\gamma^{1,2}}{1+\exp\left(0.5\frac{g}{t_o} - 1.33\right)}\right] = 1.874$$

$$k_p = 1.0 \quad \text{(tension)}$$

$$\beta = \frac{d_1 + d_2}{2d_o} = \frac{2 \cdot 60.3}{2 \cdot 114.3} = 0.528 \qquad 1.4(6)$$

$$\gamma_{Mj} = 1.0 \qquad 2.2(2)$$

$$\sin\theta_1 = \sin 30.96^\circ = 0.514$$

$$\sin\theta_2 = \sin 90^\circ = 1$$

and

$$N_{1,Rd} = \frac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1.8 + 10.2\frac{d_1}{d_0}\right)/\gamma_{M5} =$$

$$= \frac{1.874 \cdot 1 \cdot 235 \cdot 5^2}{0.514}\left(1.8 + 10.2\frac{60.3}{114.3}\right)/1.0 = 153820\text{N} =$$

$$= 153.8 \text{ kN} > 97.18 \text{ kN}$$

$$N_{2,Rd} = \frac{\sin\theta_1}{\sin\theta_2}N_{1,Rd} = \frac{0.514}{1}153.8 = 79 > 50 \text{ kN}$$

b) Punching shear failure

$$d_i = 60.3 \text{ mm} < d_o - 2t_o = 114.3 - 2 \cdot 5 = 104.3 \text{ mm}$$

and

$$N_{i,Rd} = \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_i \frac{(1+\sin\theta_i)}{(2\sin^2\theta_i)}/\gamma_{M5}$$

or:

$$N_{1,Rd} = \frac{235}{\sqrt{3}}5.0 \cdot \pi \cdot 60.3\frac{(1+0.514)}{(2 \cdot 0.514^2)}/1.0 = 368225\text{N} = 368 \text{ kN} > 97.18 \text{ kN}$$

$$N_{2,Rd} = \frac{235}{\sqrt{3}}5.0 \cdot \pi \cdot 60.3\frac{(1+1)}{(2 \cdot 1)}/1.0 = 128513 \text{ N} = 129 \text{ kN} > 50 \text{ kN}$$

Therefore joint 2 is sufficient.

Remark 15. It is assumed that the end preparation of tubular bars and the welding, are executed according to the recommendations included in EN 1090-2 (see also 6.2.4 and Fig. 6.11), and the welds should be verified (here it is omitted) according to paragraph 7.3 of EN 1993-1-8.

Remark 16. If the axial force of the lower chord coexists with a bending moment (e.g. due to the existing eccentricity e = 8.1 mm), then the criterion of paragraph 7.4.2 (2) should apply:

$$\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{|M_{ip,i,Ed}|}{M_{ip,i,Rd}}\right]^2 \leq 1.0 \qquad \text{Eq. (7.3)}$$

where

$$N_{i,Ed} = N_{3.Ed} = 200 \text{ kN}$$

$$M_{ip,3.Ed} = 200 \cdot 0.81 = 162 \text{ kNcm}$$

$$N_{3.Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{17.2 \cdot 23.5}{1.0} = 404.2 \text{ kN}$$

$$M_{ip,3.Ed} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = \frac{59.7 \cdot 23.5}{1.0} = 1403 \text{ kNcm}$$

and

$$\frac{200}{404.2} + \left(\frac{162}{1403}\right)^2 = 0.495 + 0.013 = 0.508 < 1.0.$$

9.29 Example: Welded joint of a truss consisting of bars with square hollow sections (SHS)

Verify the capacity of the welded joint shown in Fig. 9.40. if the overlap of the brace bars is λ_{ov} = 40 %, and the eccentricity $e = -20$ mm. The truss has parallel chords and the length of the diagonals is 200 cm. Steel grade S 235.

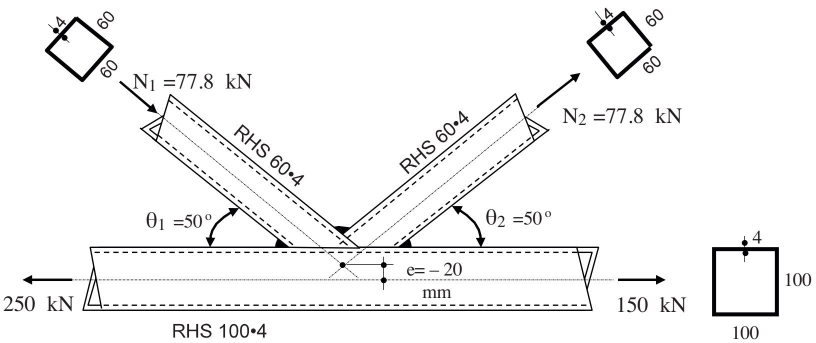


Fig. 9.40. Joint consisted of square hollow sections

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.29.1 Verification of bars

9.29.1.1 Axial forces of bars

The design axial forces that apply to the bars connected to the joint are shown in Fig. 9.40.

9.29.1.2 Cross-section classification

EN 1993-1-1. Tab. 5.2. sheet 1

Member SHS 60 · 4 (under compression)

$$\frac{c}{t} = \frac{60-8}{4} = 13 < 33\varepsilon = 33$$

Thus, the section belongs to class 1.

9.29.1.3 Verification of members

Bar 1 (SHS 60 · 4), length 200 cm and equivalent buckling length factor equal to 0.75 (EN 1993-1-1. Annex BB, 1.3.3):

$$\lambda = \frac{0.75 \cdot 200}{2.28} = 65.8$$

$$\lambda_1 = 93.9\varepsilon = 93.9$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{65.8}{93.9} = 0.70$$

EN 1993-1-1. Eq. 6.50

Buckling curve, a

EN 1993-1-1. Tab. 6.2

Imperfection factor $\alpha = 0.21$

EN 1993-1-1. Tab. 6.1

$$\Phi = 0.5[1 + 0.21(0.70 - 0.2) + 0.70^2] = 0.798$$

and $$\chi = \frac{1}{0.798 + \sqrt{0.798^2 - 0.70^2}} = 0.847$$

$$A = 8.82\ \text{cm}^2$$

$$N_{Rd} = 0.847 \cdot 8.82 \cdot 23.5/1.0 = 175.6\ \text{kN} > 77.8\ \text{kN}$$

7.2.1

Bar 2 (SHS 60· 4) (under tension)

$$N_{Rd} = 8.82 \cdot 23.5/1.0 = 207.3\ \text{kN} > 77.8\ \text{kN}$$

7.2.1

Chords SHS 100 · 4 (under tension)

7.2.1 + 5.1.5

Section area $A = 15.2\ \text{cm}^2$

$$N_{Rd} = 15.2 \cdot 23.5/1.0 = 357.2\ \text{kN} > 250\ \text{kN}$$

9.29.2 Verification of joint

The joint is an overlap type welded V-shaped joint and the bars are square hollow sections. The verification should be performed according to Table 7.10. provided that the conditions of Table 7.8 are satisfied.

9.29.2.1 Range of validity according to **Table 7.8**

The following are valid:

$$\frac{b_i}{b_o} = \frac{60}{100} = 0.60 > 0.25$$
$$\frac{b_1}{t_1} = \frac{60}{4} = 15 \text{ (class 1), (compression)}$$
$$\frac{b_2}{t_2} = \frac{60}{4} = 15 < 35 \text{ (tension)}$$
$$\frac{h_i}{b_i} = \frac{h_o}{b_o} = 1 \begin{matrix} > 0.5 \\ < 2.0 \end{matrix}$$
$$\lambda_{ov} = 40\% > 25\% \text{ and } < 100\%, \quad \text{and}$$ Fig. 1.3b
$$\frac{b_1}{b_2} = \frac{60}{60} = 1 > 0.75$$

Thus Table 7.10 of EN 1993-1-8 can be applied for this joint.

9.29.2.2 Verification of the field of application

Sections belong to class 1 (for pure bending) 7.1.2(2)

$$f_y = 235\text{N/mm}^2 < 460\,\text{N/mm}^2$$ 7.1.1(4)
$$t_i = 4\text{mm} > 2.5\text{mm}$$ 7.1.1(5)
$$t_o = 4\text{mm} < 25\text{mm}$$ 7.1.1(6)
$$\min\theta_I = 50^\circ > 30^\circ$$ 7.1.2(3)
$$e = -20\text{mm} > -0.55 d_o = -0.55 \cdot 100 = -55\text{mm}$$ 5.1.5(5)

(so, moments resulting from eccentricities may be neglected).

9.29.2.3 Design resistance according to **Table 7.10**

Brace failure for the overlapping member 2.

It is:

$$b_{eff} = \frac{10}{\frac{100}{4}} \cdot \frac{235 \cdot 4}{235 \cdot 4} \cdot 60 = 24\text{ mm} < 60\text{ mm}$$

$$b_{e,ov} = \frac{10}{\frac{60}{4}} \cdot \frac{235 \cdot 4}{235 \cdot 4} \cdot 60 = 40 \text{ mm} < 60 \text{ mm}$$

k_n = 1.0 (tension)
thus: $(25\% < \lambda_{ov} = 40\% < 50\%)$

$$N_{2,Rd} = 23.5 \cdot 0.4 \cdot \left[2.4 + 4 + \frac{40}{50} \cdot (2 \cdot 6 - 4 \cdot 0.4)\right] / 1.0 = 138.4 \text{ kN} > 77.8 \text{ kN}$$

Therefore, the joint is sufficient.

Remarks: As in Example 9.28.

9.30 Example: Bracing system of a roof

The steel warehouse shown in Fig. 9.41 with dimensions 30x50 m in plan consists of eleven (11) trusses simply supported on steel columns. Horizontal wind bracing systems are placed in the two end panels of the roof, in the level of the top chords of the trusses. The determination of the design forces that apply to the horizontal bracings is required, if the design vertical load per truss is v_{Ed} = 5 kN/m and the horizontal design load at the level of the upper chord of the truss due to wind is w_{Ed}

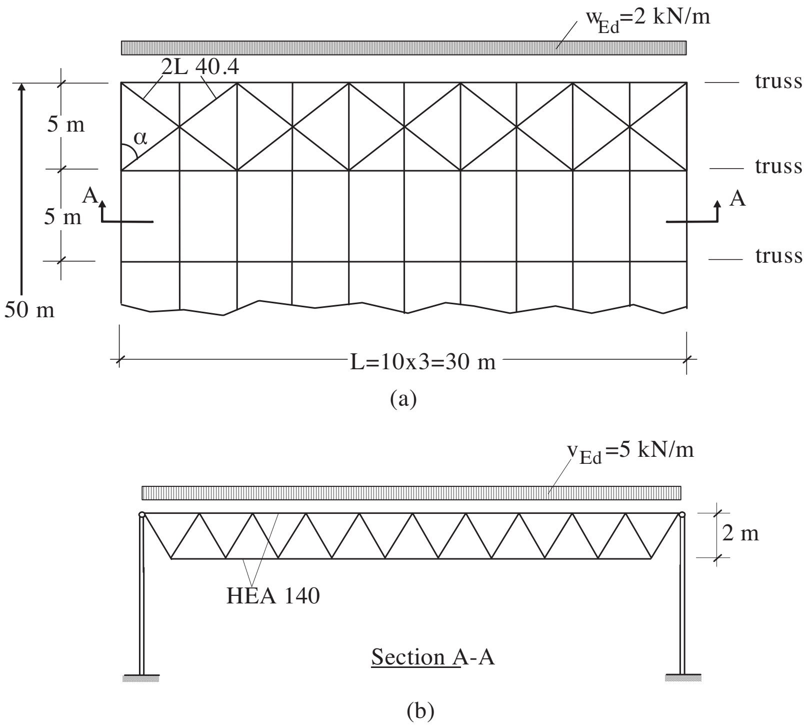


Fig. 9.41. Plan view (a) and transversal section of the warehouse (b)

= 2 kN/m. The cross-section of the truss chords is HEA 140. while the cross-section of the diagonal bracings 2L 40x4. It is assumed that the wind forces are resisted only by the windward bracing system.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.30.1 Horizontal design force

Maximum moment in a truss due to vertical load

$$M_{Ed} = v_{Ed}\frac{l^2}{8} = 5 \cdot \frac{30^2}{8} = 562.5 \text{ kNm}$$

Maximum axial force in the upper chord of a truss

$$N_{Ed} = \frac{M_{Ed}}{h} = \frac{562.5}{2} = 281.25 \text{ kN}$$ Eq. 5.14

Since there exist 11 trusses and 2 bracing systems, the number of trusses that are restrained by each bracing is m = 11 / 2 = 5.5. Fig. 5.6

Total axial compressive force to stabilize the trusses:

$$\sum N_{Ed} = 5.5N = 5.5 \cdot 281.25 = 1547 \text{ kN}$$

The initial bow imperfection is: 5.2.3(1)

$$e_0 = \alpha_m L/500 = 0.77 \cdot 3000/500 = 4.62 \text{ cm}$$ Eq. 5.12

where $\alpha_m = \sqrt{0.5\left(1+\frac{1}{m}\right)} = \sqrt{0.5\left(1+\frac{1}{5.5}\right)} = 0.77$ 5.3.3(2)

Instead of this initial bow imperfection an equivalent horizontal load q_d could be used.

The deformation δ_q in the plane of the bracing due to all horizontal loads is approximately calculated in the following, equal to 1 cm.

Then $q = \sum N_{Ed} 8 \frac{e_0 + \delta_q}{L^2} = 1547 \cdot 8 \cdot \frac{(4.62+1)\cdot 100}{3000^2} = 0.77 \text{ kN/m}$ Eq. 5.13

Total horizontal force:

$$h_{Ed} = w_{Ed} + q = 2 + 0.77 = 2.77 \text{ kN/m}$$

9.30.2 In-plane deflection of the bracing system δ_q

Bending stiffness of the bracing system:

$$EI_{eff} = 0.5EA_f h_0^2 = 0.5 \cdot 2.1 \cdot 10^4 \cdot 31.4 \cdot 500^2 = 8.24 \cdot 10^{10}\ \text{kNcm}^2 \qquad \text{Eq. 6.72}$$

where

$$A_f = 31.4\ \text{cm}^2 \quad \text{area of HEA 140.}$$

Shear stiffness of the bracing system:

If only the tension diagonal bars are active:

$$S_v = EA_d \sin^2\alpha\cos\alpha = 2.1 \cdot 10^4 \cdot 2 \cdot 3.08 \cdot \sin^2 50 \cdot \cos 50 = 48.810^3\ \text{kN}, \qquad \text{Eq. 6.9}$$

where $A_d = 2 \cdot 3.08 = 6.16\ \text{cm}^2$ cross-section area of diagonal (2L 40.4).

Inplane deflection:

$$\delta_q = \frac{5}{384} h_{\text{Ed}} \frac{l^4}{EI_{eff}} + \frac{h_{Ed} \cdot l^2}{8S_v} = \frac{5}{384} \cdot 0.0277 \cdot \frac{3000^4}{8.24 \cdot 10^{10}} + \frac{0.0277 \cdot 3000^2}{8 \cdot 48.8 \cdot 10^3} =$$
$$= 0.35 + 0.64 = 0.99 cm \approx 1.0\ \text{cm}$$

(Literature)

Therefore, the initial assumption that $\delta_q = 1.0$ cm is correct, and the total horizontal design force on the bracing system is: $h_{Ed} = 2.77$ kN/m.

9.31 Example: Vertical bracing system in single storey buildings

The longitudinal side of an industrial building is restrained in the middle panel through an X bracing system (Fig. 9.42). Verify the capacity of the members of the bracing system, if the horizontal and vertical design loads are P_d = 300 kN and H_d = 80 kN (self-weight of the steel structure is included). The columns' heads are restrained in the out of plane direction, while only the tension diagonals are considered as active. Steel grade S 235.

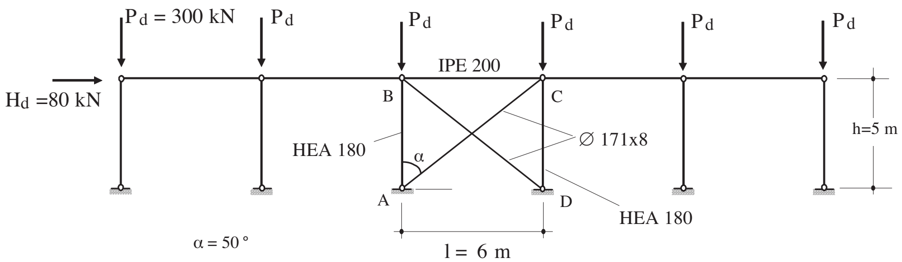


Fig. 9.42. Longitudinal side of a single storey building

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

Properties of cross-sections

$$IPE200: A_b = 28.5\ \text{cm}^2 \qquad I_b = 1940\ \text{cm}^4$$
$$HEA180: I_c = 2510\ \text{cm}^4$$
$$\emptyset 171 \times 8: \ A_d = 41\ \text{cm}^2 \quad i = 5.77\ \text{cm}$$

Length of diagonals $d = \sqrt{5^2 + 6^2} = 7.81$ m

9.31.1 Method of analysis of the structure

At first it should be determined if the analysis of the structure shall be of first or second order. The stability of the structure is ensured through the X bracing system, but since only the tension diagonal is assumed to be active, the shear stiffness of the bracing system should be calculated using only one diagonal bar.

The shear stiffness of the X bracing system, neglecting the compressive diagonals, is given from the following relation (Literature):

$$S_v = \frac{1}{\frac{1}{EA_d \cdot \sin^2\alpha \cdot \cos\alpha} + \frac{1}{EA_b \cdot \cot\alpha}} =$$
$$= \frac{1}{\frac{1}{2.1 \cdot 10^4 \cdot 41 \cdot \sin^2 50.2° \cdot \cos 50.2°} + \frac{1}{2.1 \cdot 10^4 \cdot 28.5 \cdot \cot 50.2°}} = 197 \cdot 10^3\ \text{kN}$$

The horizontal displacement at the top of the storey due to the horizontal force H_{Ed} is given from the relation:

$$\delta_{H,Ed} = \frac{H_{Ed} \cdot h}{S_v} \qquad \text{Eq. 5.2}$$

The factor of Eq. 5.2 is $\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$ where $V_{Ed} = 6 \cdot 300 = 1800$ kN = vertical load of storey.

$$\alpha_{cr} = \frac{H_{Ed}}{V_{Ed}} \frac{h}{\frac{H_{Ed}}{S_v}} = \frac{197 \cdot 10^3}{1800} = 109 > 10 \qquad \text{Eq. 5.1}$$

Thus, first order elastic analysis should be used.

9.31.2 Imperfections

In structural analysis, appropriate equivalent global (for frames and braces) and local (for individual members) imperfections should be applied. For building frames, sway imperfections may be disregarded if: 5.3.2(1)

$$H_{Ed} \geqslant 0.15 V_{Ed} \qquad \text{Eq. 5.7}$$

In this example: $H_{Ed} = 80\,\text{kN} < 0.15 \cdot 1800 = 270\,\text{kN}$, so, only global imperfections should be considered.

Global initial sway imperfections may be determined from: 5.3.2(3)

$$\phi = \phi_o \alpha_h \cdot \alpha_m \qquad \text{Eq. 5.5}$$

where ϕ_o = 1 / 200 is the basic value and α_h, α_m are the reduction factors as follows:

$$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{5}} = 0.89 \quad \text{but} \quad \frac{2}{3} < \alpha_h < 1.$$

$$\alpha_m = \sqrt{0.5\left(1+\frac{1}{m}\right)} = \sqrt{0.5\left(1+\frac{1}{6}\right)} = 0.76$$

(m is the number of columns in a row).

Finally:

$$\phi = \frac{1}{200} \cdot 0.89 \cdot 0.76 = \frac{1}{295}$$

Instead of the imperfection, an equivalent horizontal force is introduced:

$$\Delta H_d = \phi \cdot V_{Ed} = \frac{1}{295} \cdot 1800 = 6.10\,\text{kN}$$

and the total horizontal force becomes:

$$H_{Ed} = 80 + 6.10 = 86.10\,\text{kN}$$

Local bow imperfections may be neglected since there is not any moment resistant joint at the members end. 5.3.2(6)

9.31.3 Verification of diagonals

The diagonal under tension is verified.

Force applied on the diagonal

$$N_{Ed} = \frac{H_{Ed}}{\sin\alpha} = \frac{86.1}{\sin 50.2^\circ} = 112\,\text{kN}$$

Resistance of diagonal (gross section)

$$N_{Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{41 \cdot 23.5}{1.0} = 963\,\text{kN} > 112\,\text{kN} \qquad \text{Eq. 6.6}$$

9.31.4 Verification of columns

Since the columns AB and CD belong also to the X bracing system, besides the load P_d an additional load applies on them due to the horizontal force H_d. The most unfavorable column is CD since the two axial loads due to P_d and H_d are added.

Design axial load

$$N_{Ed} = P_d + H_{Ed}\frac{h}{l} = 300 + 86.1 \cdot \frac{5}{6} = 372\,\text{kN}$$

The buckling lengths are calculated for non-sway frame since the vertical bracing ensures the lateral stability of the structure. Due to the pinned joints at the ends of the columns these buckling lengths are considered equal to the column length.

y-y axis

$$\lambda_y = \frac{L_{cr}}{i_y} = \frac{500}{7.45} = 67 \qquad \text{Eq. 6.50}$$

$$\overline{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{67}{93.9} = 0.71$$

$$h/b = 171/180 < 1.2 \qquad t_f < 100\,\text{mm}, \qquad \text{Tab. 6.2}$$

buckling curve *b*

$$\chi_y = 0.778$$

z-z axis

$$\lambda_z = \frac{500}{4.52} = 111 \qquad \text{Eq. 6.50}$$

$$\overline{\lambda}_z = \frac{111}{93.9} = 1.18$$

Buckling curve *c* Tab. 6.2

$$\chi_z = 0.443$$

$$\chi = \min\{\chi_y, \chi_z\} = 0.443$$

$$N_{b,Rd} = \chi \frac{A f_y}{\gamma_{M1}} = 0.443 \cdot \frac{45.3 \cdot 23.5}{1.0} = 472\,\text{kN} \qquad \text{Eq.6.47}$$

And finally

$$N_{Ed} = 372\,\text{kN} < N_{b,Rd} = 472\,\text{kN}$$

The verification to torsional buckling of the column is omitted.

9.32 Example: Non-sway moment resisting frame

The frame shown in Fig. 9.43 is subjected to design loads q_{Ed} = 9 kN/m and H_{Ed} = 9 kN. The joints B and D are restrained in the out-of-plane direction. The members of the frame are to be checked. Steel grade S 235.

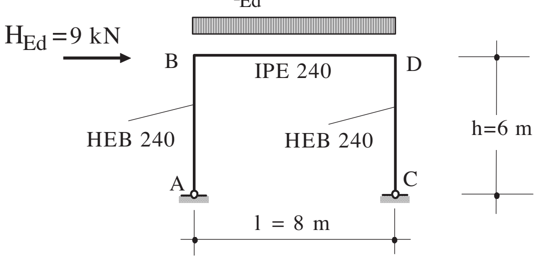


Fig. 9.43. Geometry and loading of the frame

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

Properties of cross-sections:

$$\begin{aligned}
\text{IPE 240:}\quad & A = 39.1\ \text{cm}^2, \quad I_y = I_b = 3890\ \text{cm}^4, \quad W_{pl,y} = 366 cm^3\\
& I_z = 283.6\ \text{cm}^4, \quad W_{el,y} = 324\ \text{cm}^3, \quad I_t = 12.9\ \text{cm}^4\\
& I_w = 37400\ \text{cm}^6 \quad i_y = 9.77\ \text{cm}\\
\text{HEB 240:}\quad & A = 106\ \text{cm}^2, \quad I_y = I_c = 11260\ \text{cm}^4, \quad i_y = 10.3\ \text{cm},\\
& i_z = 6.08\ \text{cm}, \quad W_{pl,y} = 1053\ \text{cm}^3, W_{el,y} = 938\ \text{cm}^3,\\
& I_z = 3920\ \text{cm}^4, \quad I_t = 103\ \text{cm}^4, I_w = 487 \cdot 10^3\ \text{cm}^6
\end{aligned}$$

9.32.1 Selection of method of frame analysis

The critical factor α_{cr} for sway buckling mode will be determined. 5.2.1(3)

$$k = \frac{I_b}{I_c}\frac{h}{l} = \frac{3890}{11260} \cdot \frac{6}{8} = 0.259$$

Horizontal displacement at the top of the frame due to a horizontal load H_{Ed} = 1 kN.

$$\delta_{H,Ed} = \frac{h^3}{12EI_c}\frac{2k+1}{k}H_{Ed} = \frac{600^3}{12 \cdot 2.1 \cdot 10^4 \cdot 11260} \cdot \frac{2 \cdot 0.259+1}{0.259} \cdot 1 = 0.446\ \text{cm}$$

Total vertical load at the top of the frame

$$V_{Ed} = q_{Ed} l = 9 \cdot 8 = 72\ \text{kN}$$

$$\alpha_{cr} = \frac{H_{Ed}}{V_{Ed}}\frac{h}{\delta_{H,Ed}} = \frac{1}{72} \cdot \frac{600}{0.446} = 18.7 > 10 \qquad \text{Eq. 5.2}$$

Check of axial compression in the beam BD:

$$\bar{\lambda}_y = \frac{L_{cr}}{i_y}\frac{1}{\lambda_1} = \frac{800}{9.97}\frac{1}{93.9} = 0.85 \qquad \text{Eq. 6.50}$$

$$N_{Ed} = 6.82 + 4.63 = 11.5\ \text{kN}$$

(see the following analysis) and

$$\bar{\lambda}_y = 0.85 < 0.3\sqrt{\frac{Af_y}{N_{Ed}}} < 0.3\sqrt{\frac{39.1 \cdot 23.5}{11.5}} = 2.68 \qquad \text{Eq. 5.3}$$

The axial compression in the beam is not significant and the approximate formula applied to determine α_{cr} may be used.

In addition, since $\alpha_{cr} > 10$. first order elastic analysis may be applied.

9.32.2 Imperfections

9.32.2.1 Imperfections of the frame

Global initial sway imperfections: 5.3.2(3)

$$\phi = \phi_o \cdot \alpha_h \cdot \alpha_m \qquad \text{Eq. 5.5}$$

where:

$$\phi_o = 1/200$$

$$\alpha_h = \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{6}} = 0.82 \quad \text{but} \quad \frac{2}{3} < a_h < 1$$

$$\alpha_m = \sqrt{0.5\left(1+\frac{1}{m}\right)} = \sqrt{0.5\left(1+\frac{1}{2}\right)} = 0.87$$

Finally:

$$\phi = \frac{1}{200} \cdot 0.82 \cdot 0.87 = \frac{1}{280}$$

Instead of this imperfection an equivalent horizontal force is introduced:

$$\Delta H_d = \phi V_{Ed} = \frac{1}{280} \cdot 72 = 0.26\,\text{kN} \qquad \text{Fig. 5.4}$$

and the total horizontal force becomes:

$$H_{Ed} = H_d + \Delta H_d = 9 + 0.26 = 9.26\,\text{kN}$$

9.32.2.2 Imperfections of members 5.3.2(6)

Local bow imperfections may be neglected since the frame is not sensitive to second order effects.

9.32.3 Frame analysis

First order elastic analysis is applied.

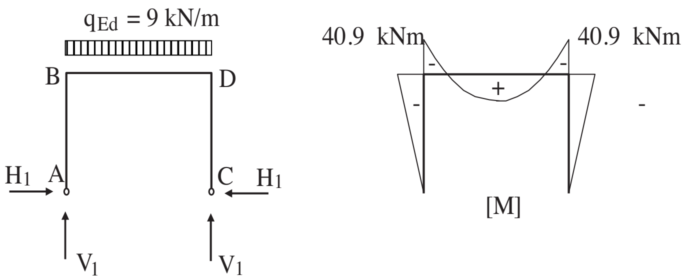


Fig. 9.44. Vertical loading and moments' diagram

$$H_1 = \frac{q_{Ed} l^2}{4h(2k+3)} = \frac{9 \cdot 8^2}{4 \cdot 6 \cdot (2 \cdot 0.259 + 3)} = 6.82 \text{ kN}$$

$$V_1 = \frac{q_{Ed} l}{2} = \frac{9 \cdot 8}{2} = 36 \text{ kN}$$

$$M_{B1} = M_{D1} = -H_1 h = -6.82 \cdot 6 = -40.9 \text{ kNm}$$

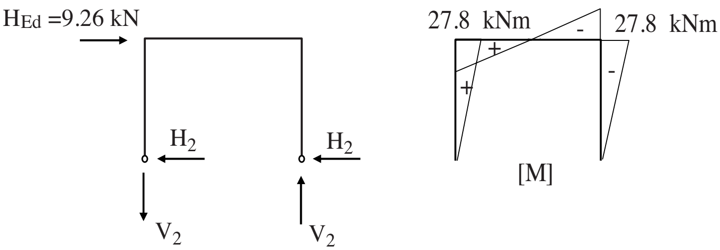


Fig. 9.45. Horizontal loading and moments' diagram

$$H_2 = \frac{H_{Ed}}{2} = \frac{9.26}{2} = 4.63 \text{ kN}$$

$$V_2 = H_{Ed} \frac{h}{l} = 9.26 \cdot \frac{6}{8} = 6.95 \text{ kN}$$

$$M_{B2} = -M_{D2} = H_2 h = 4.63 \cdot 6 = 27.8 \text{ kNm}$$

9.32.4 Verification of columns

The most unfavorable column CD is examined.

Design axial load

$$N_{Ed} = V_1 + V_2 = 36 + 6.95 = 43 \text{ kN}$$

Shear force

$$V_{Ed} = H_1 + H_2 = 6.82 + 4.63 = 11.5 \text{ kN}$$

$$M_D = -40.9 - 27.8 = -68.7 \text{ kNm}$$

$$M_C = 0$$

9.32.4.1 Flexural buckling in the plane of the frame

Buckling length 5.2.2(7b)

Since $\alpha_{cr} > 10$ the buckling length of columns is taken equal to the height of the floor.

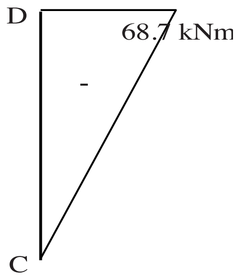


Fig. 9.46. Bending moment diagram

$$L_{\text{cr}} = 6m$$
$$\bar{\lambda}_y = \frac{L_{\text{cr}}}{i_y} \cdot \frac{1}{\lambda_1} = \frac{600}{10.3} \cdot \frac{1}{93.9} = 0.62$$
$$N_{\text{cr},y} = \frac{\pi^2 E I_y}{L_{\text{cr}}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 11260}{600^2} = 6483 \text{ kN}$$
$$h/b = 240/240 = 1 < 1.2 \quad t_f < 100 \text{ mm}$$

Buckling curve b Tab. 6.2

$$\chi_y = 0.8269$$

9.32.4.2 Flexural buckling out of plane

Since the joints B and D are restrained out-of-plane, the buckling length is equal to the story height, $L_{\text{cr}} = 6$ m.

$$\bar{\lambda}_z = \frac{600}{6.08} \cdot \frac{1}{93.9} = 1.05$$ Eq. 6.50

$$N_{\text{cr},z} = \frac{\pi^2 E I_z}{L_{\text{cr}}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 3920}{600^2} = 2257 \text{ kN}$$

buckling curve c Tab. 6.2

$$\chi_z = 0.511$$

9.32.4.3 Lateral torsional buckling

Elastic critical moment of lateral torsional buckling
Ratio of end moments $\psi = 0$.
Since there is no transverse force: $\mu_0 = 100$, $C_2 = 0$.
There is no intermediate supports: $k = 1$

$$I = \frac{1}{7} + \frac{0}{4.6} + \frac{0}{7} - \frac{1+0}{2.3+100} + \frac{0.39}{100^2} = 0.1385$$

$$C_1 = \frac{1}{\sqrt{2I}} = \frac{1}{\sqrt{2 \cdot 0.1385}} = 1.90 \quad \text{(see Chapter 4. Table 4.4)}$$

$$M_{\mathrm{cr}} = C_1 \frac{\pi^2 EI_z}{L^2}\left[\frac{I_w}{I_z} + \frac{L^2 GI_t}{\pi^2 EI_z}\right]^{0.5} =$$

$$= 1.90 \cdot \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 3920}{600^2}\left[\frac{487 \cdot 10^3}{3920} + \frac{600^2 \cdot 103}{\pi^2 2.6 \cdot 3920}\right]^{0.5} = 95195 \text{ kNcm}$$

Cross-section class 1. $W_y = W_{pl,y}$ 6.3.2.2(1)

$$\overline{\lambda}_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{\mathrm{cr}}}} = \sqrt{\frac{1053 \cdot 23.5}{95195}} = 0.51$$

Rolled cross-section, $h/b = 1 < 2$

buckling curve a Tab. 6.4

$$\chi_{LT} = 0.9211$$

9.32.4.4 Verification of stability

For comparison purposes the verification of the stability will be performed using both Methods proposed in EN93-1-1. Annex A (Method 1) and Annex B (Method 2).

Method 1 (Annex A):

$$\mu_y = \frac{1 - \frac{43.0}{6483}}{1 - 0.8269\frac{43.0}{6483}} = 0.999$$ Annex A, Tab. A.1

$$\mu_z = \frac{1 - \frac{43.0}{2257}}{1 - 0.511\frac{43.0}{2257}} = 0.990$$

$$\alpha_{LT} = 1 - \frac{I_T}{I_y} = 1 - \frac{103}{11260} = 0.99$$

$$C_{my,0} = 0.79 + 0.21 \cdot 0 + 0.36(0 - 0.33)\frac{43}{6483} = 0.79$$

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A}{W_{El,y}} = \frac{6870}{43.0} \cdot \frac{106}{938} = 18.1$$

$$C_{my} = C_{my,0} + (1 - C_{my,0})\frac{\sqrt{\varepsilon_y} \cdot \alpha_{LT}}{1 + \sqrt{\varepsilon_y}\alpha_{LT}} =$$

$$= 0.79 + (1 - 0.79)\frac{\sqrt{18.1} \cdot 0.99}{1 + \sqrt{18.1} \cdot 0.99} = 0.96$$ Annex A, Tab. A.2

Critical torsional buckling force

$$N_{cr,T} = \frac{A}{I_y + I_z}\left(GI_t + \frac{\pi^2 EI_w}{l_T^2}\right) = \frac{106 \cdot 2,1 \cdot 10^4}{11260 + 3920}\left(\frac{103}{2.6} + \frac{\pi^2 \cdot 487 \cdot 10^3}{600^2}\right) = 7767 \text{ kN}$$

$$C_{mLT} = C_{my}^2 \frac{\alpha_{LT}}{\sqrt{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \frac{N_{Ed}}{N_{cr,T}}\right)}} = 0.96^2 \frac{0.99}{\sqrt{\left(1 - \frac{43}{2257}\right)\left(1 - \frac{43}{7767}\right)}} = 0.92.$$

The interaction factors shall be determined using elastic analysis 6.3.3(5), Note 3

$$k_{yy} = C_{my} \cdot C_{mLT} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,y}}} = 0.96 \cdot 0.92 \cdot \frac{0.999}{1 - \frac{43}{6483}} = 0.888$$ Annex A, Tab. A.1

$$k_{zy} = C_{my} \cdot C_{mLT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,y}}} = 0.96 \cdot 0.92 \cdot \frac{0.999}{1 - \frac{43}{6483}} = 0.880$$

$$N_{Rk} = A \cdot f_y = 106 \cdot 23.5 = 2491 \text{ kN}$$ Tab. 6.7

$$M_{y,Rk} = W_{pl,y} \cdot f_y = 1053 \cdot 23.5 = 24745 \text{ kNcm}$$

Verification

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy} \frac{M_{y,Ed}}{\chi_{LT} \cdot M_{y,Rk}/\gamma_{M1}} = \frac{43}{0.8269 \cdot 2491/1.0} + $$

$$+ 0.888 \cdot \frac{6870}{0.9211 \cdot 24745/1.0} = 0.02 + 0.27 = 0.29 < 1$$ Eq. 6.61

$$\frac{N_{Ed}}{\chi_z N_{Rk/\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed}}{\chi_{LT} \cdot M_{y,Rk}/\gamma_{M1}} = \frac{43}{0.511 \cdot 2491/1.0} + $$

$$+ 0.880 \cdot \frac{6870}{0.9211 \cdot 24745/1.0} = 0.03 + 0.27 = 0.30 < 1$$ Eq. 6.62

Method 2 (Annex B):

The member is susceptible to torsional deformations, so the interaction factors shall be found from Table B2.

$C_{my} = 0.9$ (for a sway buckling mode, Annex B, Tab. B.3)

For $\psi = 0$ and without any transversal load (Annex B, Tab. B.3):

$$C_{mLT} = 0.6 + 0.4\psi = 0.6 > 0.4$$

$$\bar{\lambda}_y = 0.62 \quad \text{and} \quad \chi_y = 0.8269:$$

$$k_{yy} = C_{my}\left(1 + 0.6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}}\right) \leq C_{my}\left(1 + 0.6 \frac{N_{Ed}}{\chi_y \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

or $k_{yy} = 0.907 < 0.911$ and

$$k_{zy} - \left(1 - \frac{0.05\bar{\lambda}_z}{(C_{mLT} - 0.25)} \cdot \frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}}\right) \geq \left(1 - \frac{0.05}{(C_{mLT} - 0.25)} \cdot \frac{N_{Ed}}{\chi_z \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

or $k_{zy} = 1 \geq 1$. with $\bar{\lambda}_z = 1.05$ and $\chi_z = 0.511$.

Verification [Eqs. (6.61) and (6.62)]:
For $\chi_y = 0.8269$, $\chi_z = 0.511$ and $\chi_{LT} = 0.9211$:

$$\frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} = 0.02 + 0.273 = 0.293 < 1$$

$$\frac{N_{Ed}}{\chi_z \cdot N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} = 0.03 + 0.376 = 0.406 < 1$$

9.32.4.5 Cross-section verification

In addition to member verification, the verification of the cross-sections at the ends of the columns is necessary. The most unfavorable cross-section is at D. 6.3.3(2)

$$N_{Ed} = 43.0\ \text{kN} \quad \text{V}_{\text{Ed}} = 11.5\ \text{kN} \quad \text{M}_{\text{Ed}} = 68.7\ \text{kNm}$$

$$A_v = 106 - 2.24 \cdot 1.7 + (1.0 + 2 \cdot 2.1) \cdot 1.7 = 33.2\ \text{cm}^2 > \text{nh}_\text{w}\text{t}_\text{w} =$$

$$= 1 \cdot 16.4 \cdot 1 = 16.1\ \text{cm}^2$$ 6.2.6(3a)

$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{Mo} = 33.2 \cdot (23.5/\sqrt{3})/1.0 = 450\ \text{kN}$$ Eq. 6.18

$$V_{Ed} = 11.5\ \text{kN} < 0.5V_{pl,Rd} = 0.5 \cdot 450 = 225\ \text{kN}$$

Thus, it is not necessary to consider interaction between moments and shear forces. 6.2.8(2)

$$N_{pl,Rd} = Af_y/\gamma_{Mo} = 106 \cdot 23.5/1.0 = 2491\ \text{kN}$$

$$n = N_{Ed}/N_{pl,Rd} = 43.0/2491 = 0.02$$

$$M_{pl,y,Rd} = 1053 \cdot 23.5/(1.0 \cdot 100) = 247\ \text{kNm}$$

$$\alpha = \frac{A - 2bt_f}{A} = \frac{106 - 2 \cdot 24 \cdot 1.7}{106} = 0.23$$

$$M_{Ny,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5\alpha) = 247 \cdot (1-0.02)/(1-0.5 \cdot 0.23) =$$

$$= 273\ \text{kNm} > M_{pl,y,Rd}$$ Eq. 6.36

Thus:

$$M_{N,y,Rd} = 247\ \text{kNm}$$

and

$$\frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{68.7}{247} = 0.28 < 1$$ Eq. 6.41

9.32.5 Verification of beam BD

9.32.5.1 Cross-section verification

In the most unfavorable cross-section D, there exist the following:

$$M_{Ed} = 68.7\ \text{kNm} \quad N_{Ed} = 11.5\ \text{kN} \quad V_{Ed} = 43.0\ \text{kN}$$

$$A_v = 39.1 - 2 \cdot 12 \cdot 0.98 + (0.62 + 2 \cdot 1.5) \cdot 0.98 =$$
$$= 19.1 \text{ cm}^2 > 19.04 \cdot 0.62 = 11.8 \text{ cm}^2$$ 6.2.6(3a)
$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{Mo} = 19.1 \cdot (23.5/\sqrt{3})/1.0 = 259 \text{ kN}$$
$$V_{Ed} = 43.0 \text{ kN} < 0.5V_{pl,Rd} = 0.5 \cdot 259 = 129 \text{ kN}$$

Thus, it is not necessary to consider interaction between moments and shear forces. 6.2.8(2)

$$N_{pl,Rd} = Af_y/\gamma_{Mo} = 39.1 \cdot 23.5/1.0 = 919 \text{ kN}$$
$$n = N_{Ed}/N_{pl,Rd} = 11.5/919 = 0.013$$
$$M_{pl,y,Rd} = 366 \cdot 23.5/(1.00 \cdot 100) = 86.0 \text{ kNm}$$
$$\alpha = \frac{39.1 - 2 \cdot 12 \cdot 0.98}{39.1} = 0.40$$

$$M_{N,y,Rd} = 86 \cdot (1 - 0.014)/(1 - 0.5 \cdot 0.40) = 106 \text{ kNm} > \mathrm{M_{pl,y,Rd}}$$ Eq. 5.27

Thus: $M_{Ny,Rd} = 86$ kNm and

$$\frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{68.7}{86} = 0.80 < 1$$

9.32.5.2 Member verification for lateral torsional buckling

Moments at the ends of the beam:

$$M_B = -40.9 + 27.8 = -13.1 \text{ kNm} \quad M_D = -68.7 \text{ kNm}$$

Ratio of end moments:

$$\psi = \frac{-13.1}{-68.7} = 0.19$$

Transverse load factor (see section 4.3):

$$\mu_o = \frac{-13.1}{-68.7} = \frac{68,7}{9 \cdot 8^2/8} = 0.95$$ (Literature)

No intermediate support: $k = 1$

$$I = \frac{1}{7} + \frac{0.19}{4.6} + \frac{0.19^2}{7} - \frac{1+0.19}{2.3 \cdot 0.95} + \frac{1+0.19}{2.3 \cdot 0.95} + \frac{0.39}{0.95^2}$$
$$C_{1,D} = \frac{1}{\sqrt{2 \cdot 0.0768}} = 2.55 \quad C_2 = \frac{0.28656}{0.95 \cdot \sqrt{0.0768}} = 1.088$$
$$z_g = +\frac{24}{2} = +12 \text{ cm} \quad \text{(loading at the upper flange)}$$

Doubly symmetric cross-section: $z_j = 0$

Critical moment (at the end D where maximum moment exists)

$$M_{\mathrm{cr},D} = C_1 \frac{\pi^2 E I_z}{(kl)^2} \left[\sqrt{\frac{I_w}{I_z} + \frac{(kl)^2}{\pi^2} \frac{G I_t}{E I_z} + (C_2 z_g)^2} - C_2 z_g \right] = 2.55 \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 283.6}{(1 \cdot 800)^2} =$$

$$= \left[\sqrt{\frac{37400}{283.6} + \frac{(1 \cdot 800)^2}{\pi^2} \frac{12.9}{2.6 \cdot 283.6} + (1.088 \cdot 12)^2} - 1.088 \cdot 12 \right] =$$

$$= 5820\,\mathrm{kNcm}$$

The maximum bending moment of the span appears in a distance $\xi_o \cdot \lambda$ from the end B, where

$$\xi_o = \frac{\psi - 1}{8} \mu_o + \frac{1}{2} = \frac{0.19 - 1}{8} \cdot 0.95 + \frac{1}{2} = 0.404$$

The corresponding moment at this position is:

$$M(\xi_o) = -M_D \cdot \left[\frac{4}{\mu_o} (\xi_o - \xi_0^2) - \xi_o - \psi(1 - \xi_o) \right] =$$

$$= -(-68.7) \cdot \left[\frac{4}{0.95} (0.404 - 0.404^2) - 0.404 - 0.19(1 - 0.404) \right] = 34.1\ \mathrm{kNm}$$

Since this moment is less than the existing at tne end D, the previously determined M_{cr} is the critical for the beam BD.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{\mathrm{cr}}}} = \sqrt{\frac{366 \cdot 23.5}{5820}} = 1.22$$

Rolled cross-section: $\frac{h}{b} = \frac{240}{120} = 2$. Buckling curve *a*. Tab. 6.4

$$\chi_{LT} = 0.5175$$ Eq. 6.56

Bending moment resistance:

$$M_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}} = 0.5175 \cdot 366 \cdot \frac{23.5}{1.0} = 4451\ \mathrm{kNcm}$$ Eq. 6.55

and

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{68.7}{44.51} > 1.$$ Eq. 6.54

The verification is not satisfied, so the beam needs a lateral support.

For one intermediate lateral support:

$$k = \frac{1}{2} \qquad I = 0.0528 \qquad C_1 = 3.07 \qquad C_2 = 0.327$$

$$M_{\mathrm{cr}} = 18986\,\mathrm{kNcm} \qquad \bar{\lambda}_{\mathrm{LT}} = 0.67 \qquad \chi_{\mathrm{LT}} = 0.8614$$

and

$$M_{b,Rd} = 0.8614 \cdot 366 \cdot \frac{23.5}{1.0} = 7409 \text{ kNcm}$$

Check:

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{68.7}{74.1} = 0.93 < 1 \qquad \text{Eq. 6.54}$$

Remark 17. The influence of the axial force in the buckling verification of the beam BD has been ignored, since:

$$\frac{N_{Ed}}{N_{cr}} < 0.04 \qquad \text{6.3.1.2(4)}$$

For one intermediate lateral support it is:

$$N_{cr,t} = \frac{\pi^2 EI_z}{L_{cr}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 284}{400^2} = 368 \text{ kN}$$

And finally:

$$\frac{N_{Ed}}{N_{cr}} = \frac{11.5}{368} = 0.03 < 0.04$$

9.33 Example: Sway moment resisting frame

The frame shown in Fig. 9.47 is subjected to design loads $P_d = 100$ kN, $q_d = 9$ kN/m and $H_d = 9$ kN. The joints B and D are restrained in the out-of-plane direction. The members of the frame are to be checked. Steel grade S 235.

The geometry and the loading of the frame are the same as in Example 9.32. with the only difference the two vertical concentrated loads P_d, acting at the top of the columns, are added.

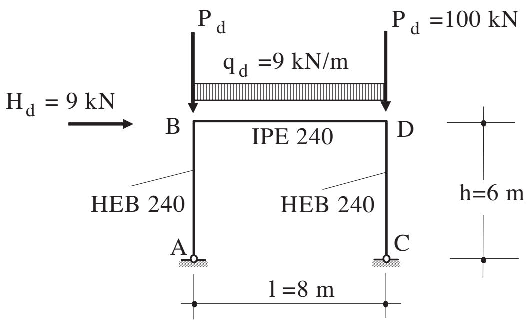


Fig. 9.47. Geometry and loading of the frame

Note. In this design example, all the references in grey through the text refer to EN 1993-1-1, unless otherwise is written.

9.33.1 Selection of method for the frame analysis

The critical factor α_{cr} for sway buckling mode will be determined. 5.2.1(3)

Since the slope of the roof is zero $\phi = 0°$, the value of α_{cr} shall be determined through Eq. (5.2). 5.2.1(4)

The horizontal displacement at the top of the frame due to a horizontal load $H_{Ed} = 1$ kN is $\delta_{H,Ed} = 0.446$ cm (Example 9.32).

Total vertical load at the top of the frame:

$$V_{Ed} = 2P_d + q_d \cdot l = 2 \cdot 100 + 9 \cdot 8 = 272 \text{ kN}$$

$$\alpha_{cr} = \frac{H_{Ed}}{V_{Ed}} \frac{h}{\delta_{H,Ed}} = \frac{1}{272} \cdot \frac{600}{0.446} = 4.9 \quad \text{Eq. 5.2}$$

The axial compression in the beam is not significant (see Example 9.32).

$$\bar{\lambda}_y < 0.3\sqrt{\frac{Af_y}{N_{Ed}}} \quad \text{Eq. 5.3}$$

It is:

$$\alpha_{cr} = 4.9 < 10$$

so, the influence of displacements must be considered in the verification of stability of the frame, since the frame is considered as sway. 5.2.2(1)

The alternative methods for global analysis and member design are as following: 5.2.2(3)

a) 2^{nd} order global analysis, accounting for local member imperfections in and out of plane of the frame. Design concerns cross-section verifications. This method is presented in paragraph 9.33.3 of this example. 5.2.2(7a)

b) 2^{nd} order global analysis, with no consideration of local member imperfections. This is followed by member design based on buckling lengths equal to the member length. This method is presented in paragraph 9.33.4 of this example. 5.2.2(7b)

c) 1^{st} order global analysis, with no concideration of local member imperfections. This is followed by member design based on column buckling lengths equal from the unbraced frame. This method is presented in paragraph 9.33.5 of this example. It will be seen that this method is inappropriate for design. 5.2.2(8)

9.33.2 Geometric imperfections

9.33.2.1 Global imperfections

The global initial sway imperfection has been determined in Example 9.32 as equal to:

$$\phi = \frac{1}{280}$$

Instead of this imperfection an equivalent horizontal force ΔH_d is introduced: Fig. 5.4

$$\Delta H_d = \varphi V_{Ed} = \frac{1}{280} 272 = 0.97 \text{ kN}$$

The total horizontal force is:

$$H_{Ed} = H_d + \Delta H_d = 9 + 0.97 = 9.97 \text{ kN}$$

9.33.2.2 Imperfections of members

Local bow imperfections in the plane of the frame may be neglected since for the most unfavorable column CD it is $\bar{\lambda}_y = 0.62$ (from Example 9.32) and $N_{Ed} = 145$ kN (from the following analysis of the frame).

Thus, the following condition is satisfied:

$$\bar{\lambda}_y < 0.5\sqrt{\frac{A \cdot f_y}{N_{Ed}}} \quad \text{or} \quad 0.62 < 0.5\sqrt{\frac{106 \cdot 23.5}{145}} = 2.07$$ Eq. 5.8

9.33.3 2nd order analysis and cross-section verification 5.2.2(7a)

In the analysis, global and local imperfections are considered.

Global imperfections have the form of an initial slope, as determined in paragr. 9.33.2.1. Instead of this slope, an equivalent horizontal force ΔH_d is applied, and the total horizontal force is $H_{Ed} = 9.97$ kN.

Local imperfections have the form of an initial bow of the members in and out of plane of the frame, to take into account flexural and torsional-flexural buckling. It is not necessary to consider member imperfections in the plane of the frame (see paragr. 9.33.2.2).

Local member bow imperfections out of plane of the frame with maximum value $k \cdot e_{o,d}$ (where $k = 0.5$) are considered, to account for lateral torsional buckling. Without LTB consideration, k would be equal to 1.0. 5.3.4(3)

The values of $e_{o,d}$ are determined as follows:

Columns:

Cross-section HEB 240

$$h/b = 1 < 1.2 \quad \text{buckling about z-z axis, } t_f < 100 \text{ mm}$$

so, the buckling curve is c. Tab. 6.2

Initial bow $e_o/L = 1/200$ for elastic analysis. Tab. 5.1

Instead of initial bow imperfections, equivalent transversal forces may apply:

$$q = \frac{8N_{Ed} \cdot e_{o,d}}{L^2} = \frac{8N_{Ed}}{200L}$$ Fig. 5.4

An initial evaluation for the axial force of the columns leads to:

$$N_{Ed} = P_d + \frac{1}{2} q_d \cdot l = 100 + 36 = 136 \text{ kN}$$

Thus, the equivalent load is

$$q = \frac{8 \cdot 136}{200 \cdot 6} = 0.907\ \text{kN/m}$$

Beam:
Cross-section IPE 240 $h/b = 2 > 1.2$ buckling about z-z axis, $t_f < 40$ mm, so the buckling curve is b.

Initial bow $e_o/L = 1/250$ for elastic analysis. Tab. 5.1

An initial evaluation for the axial force of the beam leads to:

$$N_{Ed} = 5 + 7 = 12\ \text{kN} \quad \text{(see Example 9.32. paragr. 9.32.3)}$$

and thus

$$q_d = \frac{8 \cdot 12}{250 \cdot 8} = 0.048\ \text{kN/m}$$

The loading of the frame is shown in Figure 9.48.

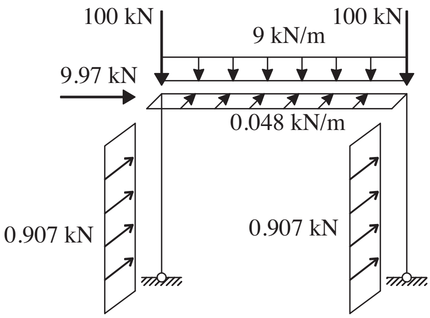


Fig. 9.48. Loading of the frame for application of 2nd order analysis

Elastic 2nd order frame analysis accompanied by cross-section design will be performed for the loading conditions illustrated in Figure 9.48. Member design is not necessary.

9.33.4 2nd order analysis and members design

5.2.2(7b)

9.33.4.1 General

Local imperfections are neglected in analysis since they are accounted for in member design. 2nd order analysis is performed so that column buckling lengths are taken equal to the floor height.

Since $\alpha_{cr} = 4.9 > 3$ second order effects are approximately taken into account through an increase of horizontal forces by the magnification factor:

$$\frac{1}{1 - \frac{1}{\alpha_{cr}}} = \frac{1}{1 - \frac{1}{4.9}} = 1.26$$

Eq. 5.4

The total horizontal force now becomes:

$$H_{Ed} = 1.26 \cdot 9.97 = 12.6\,\text{kN}$$

1st order analysis is performed, using these increased forces to account for 2nd order effects. Fig. 9.49 shows relevant diagrams M , V, N.

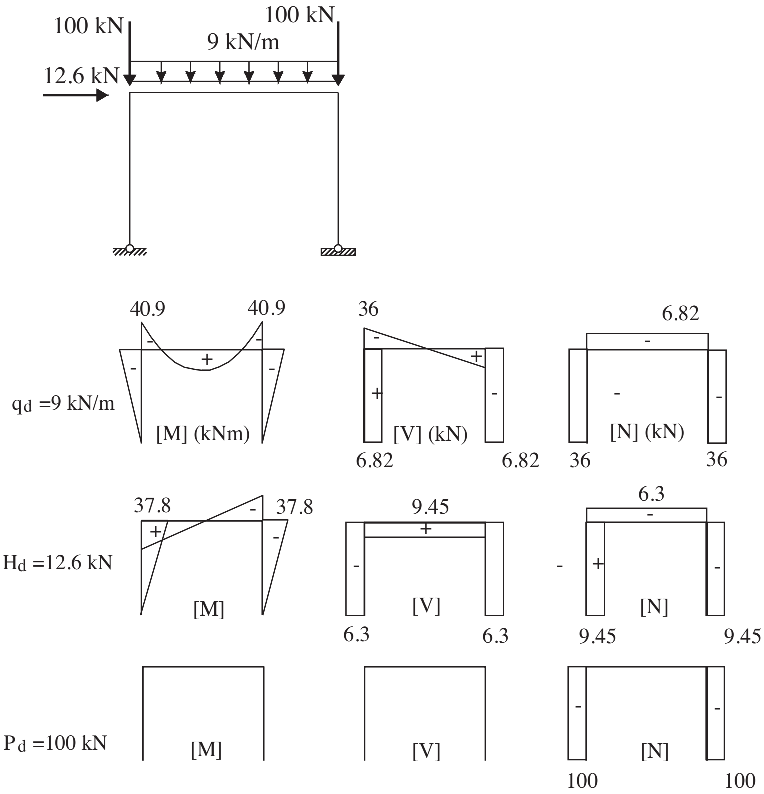


Fig. 9.49. Loading and M, V, N diagrams of the frame

9.33.4.2 Column CD verification

9.33.4.3 Cross-section design

The verification includes cross-section and member design.

The design bending moment acting at the most unfavorable joint D is:

$$M_{Ed} = M_D = -40.9 - 37.8 = -78.7\,\text{kNm}$$

The corresponding axial and shear forces of the column at joint D are:

$$N_{Ed} = 36 + 9.45 + 100 = 145.5 \text{ kN}$$
$$V_{Ed} = 6.82 + 6.3 = 13.1 \text{ kN}$$

From Example 9.32 it is:

$$V_{pl,Rd} = 450 \text{ kN}, \quad N_{pl,Rd} = 2491 \text{ kN}, \quad M_{pl,y,Rd} = 247.45 \text{ kNm}$$

Verification of shear force

$$V_{Ed} = 13.1 \text{ kN} < 0.5V_{pl,Rd} = 0.5 \cdot 450 = 225 \text{ kN}$$

and thus, the interaction between bending moments and shear forces is not taken into account. 6.2.8(2)

Verification of bending moment

$$n = N_{Ed}/N_{pl,Rd} = 145.5/2491 = 0.058$$
$$\alpha = 0.23 \quad \text{(Example 9.32)}$$
$$M_{N,y,Rd} = \frac{247 \cdot (1 - 0.058)}{1 - 0.5 \cdot 0.23} = 263 \text{ kNm} > M_{pl,y,Rd}$$ Eq. 6.36

So,

$$M_{N,y,Rd} = 247 \text{ kNm}$$

and

$$\frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{78.7}{247} = 0.32 < 1$$

9.33.4.4 Member design

The column buckling length is taken equal to the height of the floor. 5.2.2(7b)

The critical buckling loads and the moment for torsional-flexural buckling are taken (as an approximation) from Example 9.32. Moreover, to simplify the presentation of the example, the same approximation is followed for the factors C_{my} and C_{mLT}.

The difference to Example 9.32 is that the horizontal force is 12.6 kN and not 9.26 kN.

However, the differences in internal moments are generally small.

For the column CD:

$$\mu_y = \frac{1 - \frac{145.5}{6483}}{1 - 0.8269 \cdot \frac{145.5}{6483}} = 0.996$$ Annex A, Tab. A.1

$$\mu_z = \frac{1 - \frac{145.5}{2257}}{1 - 0.511 \cdot \frac{145.5}{2257}} = 0.968$$ Annex A, Tab. A.1

$$k_{yy} = 0.96 \cdot 0.92 \cdot \frac{0.996}{1 - \frac{145.5}{6483}} = 0.900$$

$$k_{zy} = 0.96 \cdot 0.92 \cdot \frac{0.968}{1 - \frac{145.5}{2257}} = 0.914$$

Member verification

$$\frac{145.5}{0.8269 \cdot 2491/1.0} + 0.900 \cdot \frac{7870}{0.9211 \cdot 24745/1.0} = 0.07 + 0.31 = 0.38 < 1$$ Eq. 6.61

$$\frac{145.5}{0.511 \cdot 2491/1.0} + 0.914 \cdot \frac{7870}{0.9211 \cdot 24745/1.0} = 0.11 + 0.32 = 0.43 < 1$$ Eq. 6.62

9.33.4.5 Verification of beam BD

The verification of beam CD is performed analogously to Example 9.32. Assuming that the resistance to lateral torsional buckling is almost equal as in Example 9.32 the verification for an intermediate lateral support is written as:

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{78.7}{74.1} = 1.06 < 1$$ Eq. 6.54

Which means that using this procedure the beam is not sufficient.

9.33.5 1st order analysis and member design 5.2.2(8)

The difference in forces in comparison to paragr. 9.33.4 is that here the horizontal force is not increased by a magnification factor and is equal to $H_{Ed} = 9.97$ kN.

Frame analysis is performed using 1st order theory. The internal forces and moments due to the loads P_d and q_d are the same as in paragr. 9.33.4. The internal forces and moments due to H_{Ed} are the same as in paragr. 9.33.4 (Fig. 9.49), divided by the ratio:

$$\frac{12.6}{9.97} = 1.26$$

The loading conditions and the internal forces and moments of the frame due to H_{Ed} are shown in Fig. 9.50.

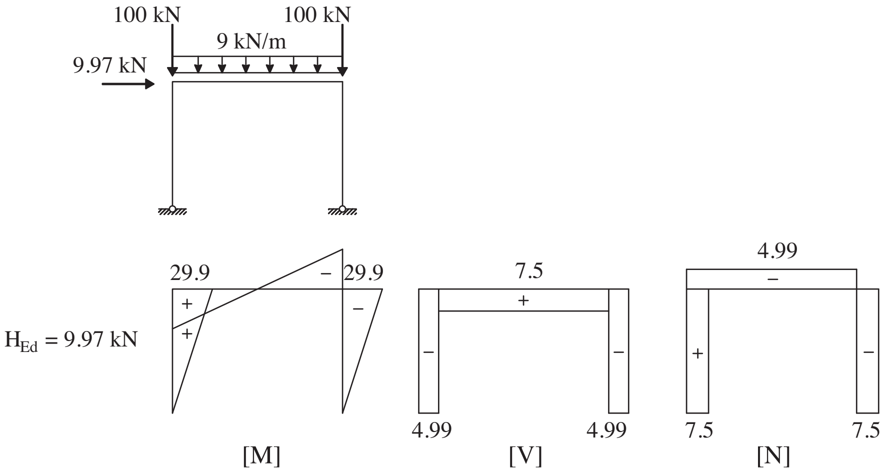


Fig. 9.50. Loading for 1st order analysis and M, V, N diagrams due to H_{Ed}

9.33.5.1 Column CD verification

9.33.5.2 Cross-section design

The design internal forces and moments at the most unfavorable joint D are:

$$M_{Ed} = 40.9 + 29.9 = 70.8 \text{ kNm}$$
$$N_{Ed} = 36 + 7.5 + 100 = 143.5 \text{ kN}$$
$$V_{Ed} = 6.82 + 4.99 = 11.81 \text{ kN}$$

In proportion to paragr. 9.33.4:

$$M_{N,y,Rd} = 247 \text{ kNm}$$

and:

$$\frac{M_{y,Ed}}{M_{N,y,Rd}} = \frac{70.8}{247} = 0.29 < 1$$

9.33.5.3 Member design

Buckling in the plane of the frame

The column buckling length is taken for a sway frame. The buckling length factors are obtained from Literature (see Example 9.17).

$$k_c = I_c/h = 11260/600 = 18.8 \text{ cm}^3$$
$$k_{11} = 1.5 I_b/l = 1.5 \cdot 3890/800 = 7.3 \text{ cm}^3$$

Upper end:

$$n_1 = \frac{k_c}{k_c + k_{11} + k_{12}} = \frac{18.8}{18.8 + 7.3} = 0.72$$

Lower end (pinned):

$$n_2 = 1.0$$

Sway frame:

$$L_{\text{cr},y} = 3.2 \cdot L = 3.2 \cdot 6 = 19.2 \text{ m}$$

$$\bar{\lambda} = \frac{L_{\text{cr}}}{i_y} \frac{1}{\lambda_r} = \frac{1920}{10.3} \cdot \frac{1}{93.9} = 1.99$$ Eq. 6.50

Buckling curve *b* (see Example 9.32) $\chi_y = 0.2113$

$$N_{\text{cr},y} = \frac{\pi^2 \cdot 2.1 \cdot 10^4 \cdot 11260}{1920^2} = 633 \text{ kN}$$

Buckling out of plane of the frame

$$L_{\text{cr}} = 6 \text{ m}$$
$$\bar{\lambda}_z = 1.05, \quad \chi_z = 0.511$$
$$N_{\text{cr},z} = 2257 \text{kN} \quad \text{(see Example 9.32)}$$

Lateral torsional buckling

$$\chi_{LT} = 0.9211 \quad \text{(see Example 9.32 and paragr. 9.33.4.2)}$$

Verification of buckling through Method 2.

$$C_{my} = 0.9 \quad \text{(for a sway frame)}$$ Annex B, Tab. B.3

For $\psi = 0$ and without any thansversal load:

$$C_{mLT} = 0.6 + 0.4\psi = 0.6 > 0.4$$ Annex B, Tab. B.3

For an I cross-section of class 1 it is:

$$k_{yy} = C_{my}\left[1 + (\bar{\lambda}_y - 0.2)\cdot\frac{N_{Ed}}{k_y \cdot N_{Rk}/\gamma_{M1}}\right]$$
$$= 0.9\left[1 + (1.99 - 0.2)\cdot\frac{143.5}{0.2113\cdot 2491/1.0}\right] = 1.34$$ Annex B, Tab. B.1

where $\bar{\lambda}_y - 0.2 = 1.99 - 0.2 = 1.79 > 0.8$.
For a column subjected to torsional-flexural buckling:

$$k_{zy} = 1 - \frac{0.1}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{x_z N_{Rk}/\gamma_{M1}}$$
$$= 1 - \frac{0.1}{(0.6 - 0.25)}\frac{143.5}{0.511\cdot 2491/1.0} = 0.968$$ Annex B, Tab. B.2

when $\bar{\lambda}_z = 1.99 > 0.4$ and

$$0.1\cdot\bar{\lambda}_z = 0.1\cdot 1.99 = 0.199 > 0.1$$

Verification:

$$\frac{N_{Ed}}{\chi_y \cdot N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} =$$
$$= \frac{143.5}{0.2113\cdot 2491/1.0} + 1.34\frac{7080}{0.9211\cdot 24745/1.0} =$$
$$= 0.27 + 0.42 = 0.69 < 1$$ Eq. 6.61

$$\frac{N_{Ed}}{\chi_z \cdot N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma + M1} =$$
$$= \frac{143.5}{0.511\cdot 2491/1.0} + 0.968\frac{7080}{0.9211\cdot 24745/1.0} =$$
$$= 0.11 + 0.30 = 0.41 < 1$$ Eq. 6.62

It is to be noticed that the two procedures of paragraphs 9.33.4 and 9.33.5 lead to similar results for out-of-plane buckling with utilization factors 0.43 and 0.41 correspondingly, but very different results for in-plane buckling where the utilization factors are 0.38 and 0.69. It may be seen that the adoption of sway buckling lengths is too conservative.

9.33.5.4 Beam verification

Assuming that the resistance to lateral torsional buckling is approximately equal as in paragr. 9.33.4.3. the verification for an intermediate support is written as:

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{70.8}{74.1} = 0.96 < 1$$

It may be seen that the method of paragraph 9.33.5 is unsafe for beam design, unless beams and joints are designed for internal moments amplified by the magnification factor of paragraph 9.33.4.

Concluding, it may be said that the method of paragraph 9.33.5 is not appropriate since it it too conservative for column design and unsafe for beam and joint design. It could be acceptable if beams and joints are designed for internal moments from 1st order analysis, duly amplified to account for 2nd order effects. However, the conservatism for column design remains and if an amplification factor should be used, why not using the method of paragraph 9.33.4.

9.34 Example: Bolted connections in tension members

Determine the design resistance of the connection shown in Fig. 9.51 for the following cases:

a) Use of M 22 bolts of class 4.6. The shear plane passes through the unthreaded portion of the bolts. Examine if in this case the ductility criterion is valid.
b) Use of M 22 preloaded bolts of class 10.9. slip resistant at the ultimate limit state. Normal holes are used, and the contact areas are classified as category B.

Steel grade of plates S 235.

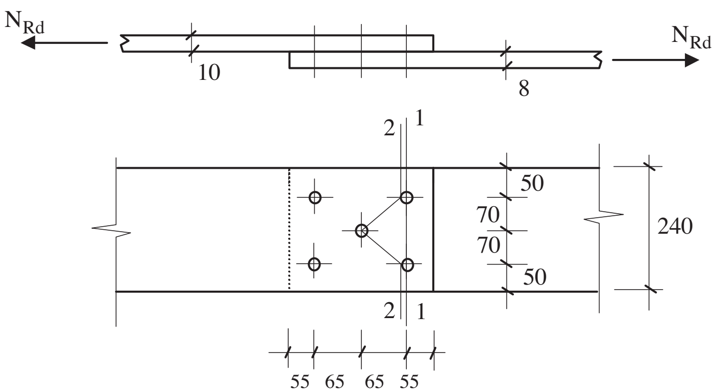


Fig. 9.51. Bolted connection of two plates

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.34.1 M 22 bolts of class 4.6

9.34.1.1 Design resistance according to tension resistance of the plates

$$N_{t,Rd} = \min\{N_{pl,Rd}, N_{uRd}\} = \min\{A f_y/\gamma_{M0}, 0.9 A_{\text{net}} f_u/\gamma_{M2}\}$$ EN 1993-1-1, 6.2.3(2)

$$A = 24 \cdot 0.8 = 19.2 \text{ cm}^2$$

Hole diameter

$$d_o = 22 + 2 = 24 \text{ mm}$$

Net area of cross-section
Cross-section 1-1 (Fig. 9.51)

$$A_{\text{net},1} = 19.2 - 2 \cdot 2.4 \cdot 0.8 = 19.2 - 3.84 = 15.36 \text{ cm}^2$$ EN 1993-1-1, 6.2.2.2(3)

Cross-section 2-2

$$A_{\text{net},2} = 19.2 - 3 \cdot 2.4 \cdot 0.8 + 2 \cdot \frac{6.5^2}{4 \cdot 7} \cdot 0.8 = 15.85 \text{ cm}^2$$ EN 1993-1-1, 6.2.2.2(4)

thus

$$A_{\text{net}} = 15.36 \text{ cm}^2$$

and

$$N_{t,Rd} = \min\{19.2 \cdot 23.5/1.0, 0.9 \cdot 15.36 \cdot 36/1.25\} = \\ = \min\{451.2, 398.1\} = 398.1 \text{ kN}$$

9.34.1.2 Design resistance based on the resistance of bolts

Shear resistance

$$F_{v,Rd} = \alpha_v f_{ub} A/\gamma_{M2} \quad \text{(per bolt)}$$ Tab. 3.4

$$A = \pi \cdot 2.2^2/4 = 3.80 \text{ cm}^2$$

Total shear resistance of bolts

$$F_{v,Rd} = 5 \cdot 0.6 \cdot 40 \cdot 3.80/1.25 = 364.8 \text{ kN}$$

Bearing resistance

$$F_{b,Rd} = \frac{k_1 a_b f_u d \cdot t}{\gamma_{M2}} \quad \text{(per bolt)}$$

And thus:

a. in the direction of the load transfer:
for end bolts:

$$\alpha_d = \frac{e_1}{3d_o} = \frac{55}{3 \cdot 24} = 0.764$$

Fig. 3.1

for inner bolts:

$$\alpha_d = \frac{p_1}{3_{do}} - \frac{1}{4} = \frac{130}{3 \cdot 24} - \frac{1}{4} = 1.555$$

$$\alpha_b = \min\left(\alpha_d, \frac{f_{ub}}{f_u}, 1,0\right) = \min\left(0.764, \frac{40}{36}, 1.0\right) = 0.764$$

b. perpendicular to the direction of load transfer:
for edge bolts:

$$= \min\left\{2.8\frac{e_2}{d_0} - 1.7, 2.5\right\} = \min\left(2.8\frac{40}{24} - 1.7, 2.5\right) = \min(4.13, 2.5) = 2.5$$

For inner bolts:

$$k_1 = \min\left\{1.4\frac{p_2}{d_0} - 1.7, 2.5\right\} = \min\left(1.4\frac{140}{24} - 1.7, 2.5\right) = \min(6.46, 2.5) = 2.5$$

Therefore, total bearing resistance for five bolts is:

$$F_{b \cdot Rd} = 5 \cdot 2.5 \cdot 0.764 \cdot 36 \cdot 2.2 \cdot 0.8/1.25 = 484.1 \text{ kN}$$

9.34.1.3 Connection design resistance

The design resistance of the connection in the examined case of common bolts is:

$$N_{Rd} = \min(N_{t,Rd}, F_{v,Rd}, F_{b,Rd}) = \min(398.1, 364.8, 484.1) = 364.8 \text{ kN}$$

9.34.1.4 Ductile behavior criterion

6.2.3(3)

The criterion is satisfied if:

$$N_{pl,Rd} \leqslant N_{uRd}$$

i.e.

$$0.9\frac{A_{\text{net}}}{A} \geqslant \frac{f_y}{f_u}\frac{\gamma_{M2}}{\gamma_{M0}}$$

It is:

$$0.9A_{\text{net}}/A = 0.9 \cdot 15.36/19.2 = 0.720 \quad \text{and}$$

$$f_y\gamma_{M2}/(f_u\gamma_{M0}) = 23.5 \cdot 1.25/(36 \cdot 1.0) = 0.816$$

thus, this criterion is not satisfied (a ductile behavior is required, for instance, in the bolted connections of critical structural members in seismic areas).

9.34.2 Preloaded bolts

9.34.2.1 Design resistance based on the resistance of the steel plates in tension

$$N_{\mathrm{net},Rd} = A_{\mathrm{net}} f_y / \gamma_{M0} = 15.36 \cdot 23.5/1.0 = 361.0\ \mathrm{kN}$$ 6.2.3(4)

9.34.2.2 Design slip resistance of bolts 3.9

Design slip resistance

Tensile stress area of a bolt M 22, $A_s = 3.03\ \mathrm{cm}^2$.
Preloading force

$$F_{p,c} = 0.7 f_{ub} A_s = 0.7 \cdot 100 \cdot 3.03 = 212.1\ \mathrm{kN}$$ Eq. 3.7

Design slip resistance per bolt

$$F_{s,Rd} = k_s \eta \mu F_{p,c} / \gamma_{M3}$$ Eq. 3.6

in which:

$k_s = 1.0$ (normal holes) Tab. 3.6
$\eta = 1.0$ (number of the friction surfaces)
$\mu = 0.4$ (slip factor) Tab. 3.7
$\gamma_{M3} = 1.10$

and finally:

$$F_{s,Rd} = 5 \cdot 1.0 \cdot 1.0 \cdot 0.4 \cdot 212.1/1.10 = 385.6\ \mathrm{kN}$$

Bearing resistance (as in paragraph 9.34.1.2) Tab. 3.2

$$F_{b,Rd} = 484.1\ \mathrm{kN}$$

9.34.2.3 Connection design resistance

The design resistance of the connection in the case of preloaded bolts is:

$$N_{Rd} = \min(N_{\mathrm{net},Rd}, F_{s,Rd}, F_{b,Rd}) = \min(361.0, 385.6, 484.1) = 361.0\ \mathrm{kN}$$

9.35 Example: Tension member splice

In the single storey frame (ABCDE) shown in Fig. 9. 9.52a, verify the tie-beam BD and its splice at an intermediate position S. The cross-section of the tie-beam is IPE 300 and the continuity at S is obtained through web and flange plates (Figs. 9.52b and c). The bolts used are of class 4.6, M22 for the flanges and M16 for the web. The design tension force transmitted by the tie-beam is $S_{Ed} = 900$ kN and the steel grade S 235.

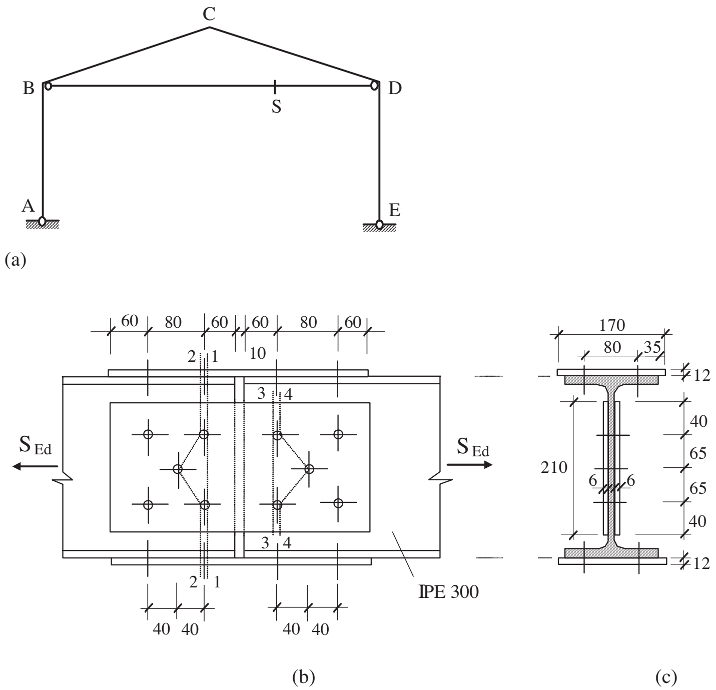


Fig. 9.52. Tension member splice

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.35.1 Tie-beam in tension

Net area of the cross section EN 1993-1-1. 6.2.2.2
Cross-section 1-1

$$A_{\text{net},1} = 53.8 - 4 \cdot 2.4 \cdot 1.07 - 2 \cdot 1.8 \cdot 0.71 = 40.97 \text{ cm}^2$$

Cross-section 2-2

$$A_{\text{net},2} = 53.8 - 4 \cdot 2.4 \cdot 1.07 - 3 \cdot 1.8 \cdot 0.71 + 2 \cdot 4^2 \cdot 0.71/4 \cdot 6.5 = 40.57 \text{ cm}^2$$

Therefore:

$$A_{\text{net}} = \min\{A_{\text{net},1}, A_{\text{net},2}\} = 40.57 \text{ cm}^2$$

Design resistance to tension EN 1993-1-1. 6.2.3

$$\begin{aligned} N_{t,Rd} &= \min\{N_{pl,Rd}, N_{u,Rd}\} = \min\{A f_y/\gamma_{M0}, 0.9 A_{\text{net}} f_u/\gamma_{M2}\} = \\ &= \min\{53.8 \cdot 23.5/1.0, 0.9 \cdot 40.57 \cdot 36/1.25\} = \\ &= \min\{1264.3, 1051.6\} = 1051.6 \text{ kN} \end{aligned}$$

and

$$S_{Ed} = 900 \text{ kN} < N_{t,Rd} = 1051.6 \text{ kN}$$

9.35.2 Distribution of the design tension force between the flanges and the web

The distribution of the total tension force between the web and the flanges (S_{Edw} and S_{Edf}) is obtained in proportion to the corresponding cross-section areas of the web and flange plates used in the splice (i.e. A_w and A_f). It is preferable to choose the ratio of these areas almost equal to the ratio of the corresponding areas of the spiced beam. It is:

$$\begin{aligned} A_f &= 2 \cdot 17 \cdot 1.2 = 40.8 \text{ cm}^2 \\ A_w &= 2 \cdot 21 \cdot 0.6 = 25.2 \text{ cm}^2 \\ S_{Edf} &= S_{Ed} \frac{A_f}{A_f + A_w} = 900 \cdot \frac{40.8}{40.8 + 25.2} = 556.4 \text{ kN} \\ S_{Edw} &= S_{Ed} \frac{A_w}{A_f + A_w} = 900 \cdot \frac{25.2}{40.8 + 25.2} = 343.6 \text{ kN} \end{aligned}$$

9.35.3 Flange plates in tension

$$A_{\text{net}} = 17 \cdot 1.2 - 2 \cdot 2.4 \cdot 1.2 = 14.64 \text{ cm}^2$$ EN 1993-1-1. 6.2.2.2

$$\begin{aligned} N_{t,Rd} &= \min\{17 \cdot 1.2 \cdot 23.5/1.0, 0.9 \cdot 14.64 \cdot 36/1.25\} = \\ &= \min\{479.4, 379.5\} = 379.5 \text{ kN} > \frac{1}{2} S_{Edf} = 278.2 \text{ kN} \end{aligned}$$ EN 1993-1-1. 6.2.3

9.35.4 Verification of flanges' bolts

Design resistance in shear, assuming that the shear plane passes through the unthreaded portion of the bolt's shaft:

$$\begin{aligned} F_{v,Rd} &= n \frac{a_v f_{ub} A}{\gamma_{M2}} = 4 \cdot \frac{0.6 \cdot 40 \cdot \pi \cdot 2.2^2}{4 \cdot 1.25} = \\ &= 291.9 \text{ kN} > \frac{1}{2} \cdot 556.4 = 278.2 \text{ kN} \end{aligned}$$ Tab. 3.4

Bearing resistance

$$a_b = \min\left\{\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u} 1.0\right\} =$$
$$= \min\left\{\frac{60}{3\cdot 24}, \frac{80}{3\cdot 24} - \frac{1}{4}, \frac{40}{36}, 1.0\right\} = 0.833$$
$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 2.5\right\} = \min\left\{2.8\frac{35}{24} - 1.7, 2.5\right\} = 2.38$$
$$F_{b,Rd} = n\frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} = 4\cdot\frac{2.38\cdot 0.833\cdot 36\cdot 2.2\cdot 1.07}{1.25} =$$
$$= 537.6\text{ kN} > \frac{1}{2}\cdot 556.4 = 278.2\text{ kN}$$

9.35.5 Web plates

Net area EN 1993-1-1. 6.2.2.2

Cross-section 3-3

$$A_{\text{net},3} = 2\cdot(21\cdot 0.6 - 2\cdot 1.8\cdot 0.6) = 20.88\text{ cm}^2$$

Cross-section 4-4

$$A_{\text{net},4} = 2\cdot[21\cdot 0.6 - 3\cdot 1.8\cdot 0.6 + 2\cdot 4^2\cdot 0.6/(4\cdot 6.5)] = 20.2\text{ cm}^2$$

Therefore:

$$A_{net} = \min\{A_{net,3}, A_{net,4}\} = 20.2\text{ cm}^2$$

and

$$N_{t,Rd} = \min\{2\cdot 21\cdot 0.6\cdot 23.5/1.0, 0.9\cdot 20.2\cdot 36/1.25\} =$$
$$= \min\{592.2, 523.6\} = 523.6\text{ kN} > S_{Edw} = 343.6\text{ kN}$$

EN 1993-1-1. 6.2.3

9.35.6 Verification of the web bolts

Shear resistance of bolts (two shear planes)

$$F_{v,Rd} = 5\cdot 2\cdot\frac{0.6\cdot 40\cdot\pi\cdot 1.6^2}{4\cdot 1.25} = 386\text{ kN} > S_{Edw} = 343.6\text{ kN}$$

Tab. 3.4

Bearing resistance

$$\alpha_b = \min\left\{\frac{60}{3\cdot 18}, \frac{80}{3\cdot 18} - \frac{1}{4}, \frac{40}{36}, 1.0\right\} = 1.0$$
$$k_1 = \min\left(2.8\frac{40}{18} - 1.7, 2.5\right) = 2.5$$

and

$$F_{b,Rd} = 5\cdot\frac{2.5\cdot 1.0\cdot 36\cdot 1.6\cdot 0.71}{1.25} = 409.0\text{ kN} > S_{Edw} = 343.6\text{ kN}$$

9.35.7 Spacings (minimum and maximum) and distances (end and edge) of bolts

9.35.7.1 Bolts of flanges

3.5

Distance from the end of plates measured in the direction of the load transfer

Tab.3.3. Fig. 3.1

Minimum distance

It is

$$e_1 = 60 \text{ mm}$$

and

$$e_1 > 1.2d_0 = 1.2 \cdot 24 = 28.8 \text{ mm}$$

In case of steel exposed to the weather or other corrosive influences, there is a limit to the maximum value of e_1. It is:

$$e_1 = 60 < 40 \text{ mm} + 4t = 40 + 4 \cdot 10.7 = 82.8 \text{ mm}$$

where t is the thickness of the thinner outer connected part, i.e. in this case the flange thickness of a IPE 300 is $t = 10.7$ mm.

Spacing of bolts measured in the direction of load transfer Tab. 3.3

Minimum spacing:

$$p_1 = 80 \text{ mm}$$

and

$$p_1 > 2.2d_0 = 52.8 \text{ mm},$$

Maximum spacing

$$p_1 = 80mm < \min(200 \text{ mm}, 14t) = \min(200, 14 \cdot 10.7) = 149.8 \text{ mm}$$

irrespectively of the corrosive influences.

Distance to the adjacent edge measured transversally to the direction of the load transfer

$$e_2 = 35 \text{ mm} \quad \text{(in IPE 300 section)}$$

and:

$$e_2 > 1.2 \cdot 24 = 28.8 \text{ mm}$$

In case of connection exposed to the weather or other corrosive influences the limit for the maximum distance referred in paragr. 7.1.1 of EN 1993-1-8 applies.

Spacing measured perpendicular to the load transfer direction between adjacent lines of bolts

Tab. 3.3

Minimum spacing

$$p_2 = 80 \text{ mm}$$

and

$$p_2 > 2.4d_0 = 2.4 \cdot 24 = 57.6 \text{ mm}$$

Maximum spacing

$$p_2 < \min(200 \text{ mm}, 14t) = 149.8 \text{ mm}$$

irrespectively of the corrosive influences.

9.35.7.2 Bolts in the web

Tab. 3.3 + Fig. 3.1

End distance e_1

$$e_1 = 60 \text{ mm}$$

and

$$e_1 > 1.2d_o = 1.2 \cdot 18 = 21.6\text{mm}$$

Additionally, in case of corrosive environment:

$$e_1 < 40 + 4 \cdot t = 40 + 4 \cdot 6 = 64 \text{ mm}$$

where t is the thickness of the thinner outer connected part.

Spacing p_1

$$p_1 = 80 \text{ mm}$$

and

$$p_1 > 2.2d_0 = 2.2 \cdot 18 = 39.6 \text{ mm}$$
$$p_1 < \min(200 \text{ mm}, 14t) = \min(200, 14 \cdot 6) = 84 \text{ mm}$$

Edge distance e_2

$$e_2 = 40 \text{ mm}$$

and

$$e_2 > 1.2d_0 = 1.2 \cdot 18 = 21.6 \text{ mm}$$

In corrosive environment it should be additionally:

$$e_2 < 40 \text{ mm} + 4t = 40 + 4 \cdot 6 = 64 \text{ mm}$$

Spacing p_2 Fig. 3.1b

$$p_2 = 65 \text{ mm} \quad \text{and for the minimum spacing:}$$
$$p_2 > 1.2d_0 = 1.2 \cdot 18 = 21.6 \text{ mm} \quad \text{and}$$
$$L = \sqrt{65^2 + 40^2} = 76.3 \text{ mm} > 2.4d_0 = 43.2 \text{ mm}$$

For the maximum spacing:

$$p_2 < \min(200 \text{ mm}, 14 \cdot 6) = 84 \text{ mm}$$

Remark 18. Minimum and maximum spacing, end and edge distances for structures subjected to fatigue should be calculated considering the provisions of EN 1993-1-9.

The local buckling resistance of the plate in compression between adjacent fasteners should be calculated according to EN 1993-1-1 using $0.6p_i$ as buckling length (EN 1993-1-8. Tab. 3.3).

9.36 Example: Angles connected through one leg

Determine the design resistance $N_{t,Rd}$ to tension of the two angles L70.7 shown in Fig. 9.53 connected to a gusset plate through one leg by a single row of bolts. Steel grade S 275, bolts M20.

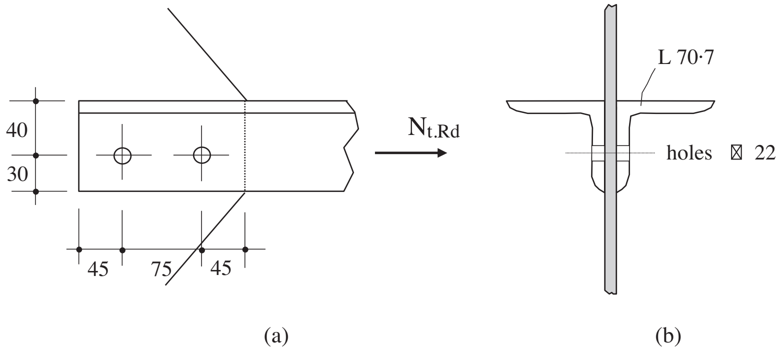


Fig. 9.53. Angles connected to a gusset plate through one leg by a single row of bolts

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.36.1 Design tension resistance of the cross-section

$$N_{pl,Rd} = A\, f_y/\gamma_{M0} = 2 \cdot 9.40 \cdot 27.5/1.0 = 517 \text{ kN}$$ EN 1993-1-1. 6.2.3

9.36.2 Reduced design ultimate resistance due to the eccentric connection

The design ultimate resistance is reduced to consider the eccentricity of the connection.

Hole diameter $d_o = 22$ mm

Net area

$$A_{\text{net}} = 2 \cdot (9.40 - 2.2 \cdot 0.7) = 15.72\ \text{cm}^2$$ 6.2.2.2

Design ultimate resistance

$$N_{u,Rd} = \beta_2 A_{\text{net}} f_u / \gamma_{M2} = 0.51 \cdot 15.72 \cdot 43/1.25 = 275.8\ \text{kN}$$ Eq. 3.12

where the factor $\beta_2 = 0.51$ is calculated by linear interpolation for $p_1 = 75$ mm $= 3.41\text{d}_\text{o}$. Tab. 3.8 + Fig. 3.9

9.36.3 Design resistance

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 275.8\ \text{kN}$$ EN 1993-1-1. 6.2.3

9.37 Example: Bolted connection under tension and shear

Verify the bolted connection of the tension member CB to the flange of a column (Fig. 9.54) for a design load $P_{Ed} = 300$ kN. The thickness of the column flange is 20 mm, the bolts are M 20 class 8.8 and the steel grade S 355.

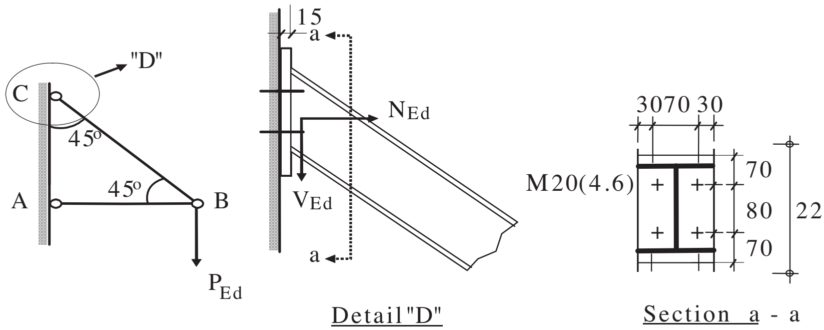


Fig. 9.54. Bolted connection under tension and shear

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.37.1 Design force in the tension member

$$F_{Ed} = P_{Ed}\sqrt{2} = 424.3 \text{ kN}$$

This force is analysed in two components (i.e. shear and tension force) with the following values:

$$N_{Ed} = V_{Ed} = F_{Ed}/\sqrt{2} = 300 \text{ kN}$$

Thus, the tension force per bolt is:

$$F_{t,Ed} = 300/4 = 75 \text{ kN}$$

and the shear force:

$$F_{v,Ed} = 300/4 = 75 \text{ kN}$$

9.37.2 Design tension resistance per bolt

$$F_{t,Rd} = k_2 f_{ub} A_s/\gamma_{M2} = 0.9 \cdot 80 \cdot 2.45/1.25 = 141 \text{ kN} \quad \text{and}$$ Tab. 3.4

$$F_{t,Ed} = 75 \text{ kN} < F_{t.Rd} = 141 \text{ kN}$$ Tab. 3.2

9.37.3 Design shear resistance per bolt (the shear plane passes through the unthreaded portion of the bolt)

$$F_{v,Rd} = \alpha_v f_{ub} A/\gamma_{M2} = 0.6 \cdot 80 \cdot \left(\pi \cdot \frac{2^2}{4}\right)/1.25 = 121 \text{ kN} \quad \text{and}$$ Tab. 3.4

$$F_{v,\mathrm{Ed}} = 75 \text{ kN} < F_{v,Rd} = 121 \text{ kN}$$ Tab. 3.2

9.37.4 Combined shear and tension

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} = \frac{75}{121} + \frac{75}{1.4 \cdot 141} = 0.62 + 0.38 = 1.0 \leqslant 1.0$$ Tab. 3.4

9.37.5 Design bearing resistance

$$F_{b.Rd} = k_1 \alpha_b f_u d t/\gamma_{M2}$$ Tab. 3.4

in which

$$\alpha_b = \min\left\{\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1.0\right\} = $$
$$= \min\left\{\frac{70}{3 \cdot 22}, \frac{80}{3 \cdot 22} - \frac{1}{4}, \frac{80}{51}, 1.0\right\} = 0.962$$ Fig. 3.1

and

$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5\right\} =$$

$$= \min\left\{2.8\frac{30}{22} - 1.7, 1.4\frac{70}{22} - 1.7, 2.5\right\} = 2.118$$

Fig. 3.1

Therefore,

$$F_{b,Rd} = 2.118 \cdot 0.962 \cdot 51 \cdot 2.0 \cdot 1.5/1.25 = 249.4 \text{ kN} \quad \text{and}$$

$$F_{v,\text{Ed}} = 75 \text{ kN} < \text{F}_{\text{b.Rd}} = 249.4 \text{ kN}$$

9.37.6 Punching shear resistance of the connected plates

Tab. 3.4

The verification is based on the relation:

$$F_{t,Ed} \leqslant B_{p.Rd} = 0.6\pi d_m t_p f_u/\gamma_{M2}$$

Tab. 3.2

in which d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller. 1.4

t_p is the thickness of the plate under the bolt or the nut. 1.4

In this example for the bolt's head M 20 the following are valid (according to DIN 6914):

- Diameter of circumscribed circle $e = 36.9$ mm.
- Diameter of inscribed circle $s = 32$ mm.
- Mean diameter $d_m = (e+s)/2 = 34.45$ mm.
- For the nut: (according to DIN 6915).
- Diameter of circumscribed circle $e = 36.9$ mm.
- Diameter of incribed circle $d_2 = 30$ mm.
- Mean diameter $d_m = (e+d_2)/2 = 33.45$ mm.

Therefore,

$$B_{p \cdot Rd} = 0.60\pi \cdot 3.345 \cdot 1.5 \cdot 51.0/1.25 = 385.9 \text{ kN}$$

and

$$F_{t \cdot Ed} = 75 \text{ kN} < B_{p \cdot Rd} = 385.9 \text{ kN}$$

Remark 19. Besides the bolts, the connection between the tension member and the column should be checked considering all joint elements (end plate thickness, column flange thickness etc.) according to EN 1993-1-8.

9.38 Example: Connection using preloaded bolts under shear and tension

The tension member CD of the structure (ABCD) shown in Fig. 9.55 is connected to the column ABC through four preloaded bolts of class 8.8. placed in normal holes. The class of friction surfaces is taken as A (EN 1993-1-8. Tab. 3.7). The force P consists of permanent 50 kN and imposed load 100 kN. The steel grade is S 235. Verify the adequacy of the bolted connection in the following cases:

1. Connection category C with M 24 bolts (slip-resistant at the ultimate limit state).
2. Connection category B with M 20 bolts (slip-resistant at the serviceability limit state).

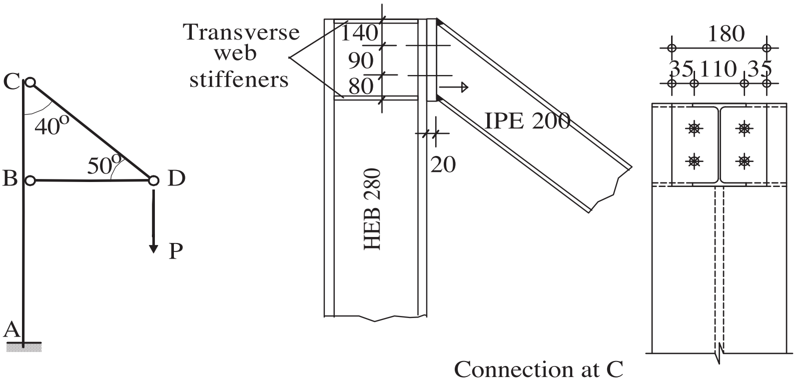


Fig. 9.55. Connection using preloaded bolts under shear and tensile forces

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.38.1 Connection category C with M 24 bolts (slip-resistant at the ultimate limit state)

9.38.1.1 Actions

Design force

$$P_{Ed} = 1.35 \cdot 50 + 1.50 \cdot 100 = 217.5 \text{ kN}$$

Design tension force in member CD

$$Z_{Ed} = P_{Ed} / \sin 50^\circ = 283.9 \text{ kN}$$

This force leads to the following components:

$$F_{v.Ed} = \frac{1}{4} Z_{Ed} \cdot \cos 40^\circ = 54.4 \text{ kN} \quad \text{(slip force per bolt)}$$

$$F_{t.Ed} = \frac{1}{4} Z_{Ed} \cdot \sin 40^\circ = 45.6 \text{ kN} \quad \text{(tension force per bolt)}$$

9.38.1.2 Design slip resistance

Preloading force:

$$F_{p,c} = 0.7 f_{ub} A_s = 0.7 \cdot 80 \cdot 3.53 = 197.7 \text{ kN} \qquad \text{Eq. 3.7}$$

(For a bolt M 24. tensile stress area $A_s = 3.53 \text{ cm}^2$).

The design slip resistance per bolt is given by the relation:

$$F_{s,Rd} = \frac{k_s n \mu (F_{p,c} - 0.8F_{t,Ed})}{\gamma_{M3}}$$ Eq. 3.8b

where

$$k_s = 1.0 \quad \text{(normal holes)}$$ Tab. 3.6

$$n = 1 \quad \text{(number of the friction surfaces)}$$

$$\mu = 0.5 \quad \text{(slip factor for friction surface category A)}$$ Tab. 3.7

$$\gamma_{M3} = 1.25$$ Tab. 2.1

Therefore

$$F_{s,Rd} = \frac{1.0 \cdot 1 \cdot 0.50 \cdot (197.7 - 0.8 \cdot 45.6)}{1.25} =$$
$$= 64.5 \text{ kN} > F_{v.Ed} = 54.4 \text{ kN}$$

9.38.1.3 Tension resistance of bolts

It should be:

$$F_{t,Ed} \leqslant F_{t,Rd}$$ Tab. 3.2

It is (per bolt):

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 80 \cdot 3.53}{1.25} =$$ Tab. 3.4
$$= 203.3 \text{ kN} > F_{t,Ed} = 45.6 \text{ kN}$$

9.38.1.4 Punching shear resistance of plates Tab. 3.4

In correlation to Example 9.37. it is taken for a M 24 bolt:
head of the bolt

$$e = 47.3 \text{ mm}, \quad s = 41 \text{ mm} \quad \text{and} \quad d_m = 44.15 \text{ mm}$$

nut

$$e = 47.3 \text{ mm}, \quad d_2 = 39 \text{ mm} \quad \text{and} \quad d_m = 43.15 \text{ mm}$$

Therefore,

$$B_{p \cdot Rd} = 0.6\pi d_m t_p f_u / \gamma_{M2} = 0.6\pi \cdot 4.315 \cdot 1.8 \cdot 36/1.25 = 421.6 \text{ kN}$$

and

$$F_{t \cdot Ed} = 45.6 \text{ kN} < B_{p \cdot Rd} = 421.6 \text{ kN}$$ Tab. 3.2

9.38.1.5 Bearing resistance

Tab. 3.4

$$\alpha_b = \min\left\{\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1.0\right\} =$$

Fig. 3.1

$$= \min\left\{\frac{140}{3\cdot 26}, \frac{90}{3\cdot 26} - \frac{1}{4}, \frac{80}{36}, 1.0\right\} = 0.904$$

$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5\right\} =$$

$$= \min\left\{2.8\frac{35}{26} - 1.7, 1.4\frac{110}{26} - 1.7, 2.5\right\} = 2.07$$

$$F_{b,Rd} = m\frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} = 4\cdot\frac{2.07\cdot 0.904\cdot 36\cdot\ 2.4\cdot 1.8}{1.25} =$$

$$= 931.3\ \text{kN} > mF_{v.Ed} = 217.5\ \text{kN}$$

($m = 4$ is the number of bolts).

9.38.2 Connection category B with M 20 bolts (slip-resistant at the serviceability limit state)

9.38.2.1 Actions

Action at serviceability

$$P_{\text{ser}} = 1.0\cdot 50 + 1.0\cdot 100 = 150\ \text{kN}$$

tension force of member CD

$$Z_{\text{ser}} = 150/\sin 50^\circ = 195.8\ \text{kN}$$

This force leads to the following components:

$$F_{v.Ed.\text{ser}} = \frac{1}{4}Z_{\text{ser}}\cos 40^\circ = 37.5\ \text{kN} \quad \text{(slip force per bolt)}$$

$$F_{t.Ed.\text{ser}} = \frac{1}{4}Z_{\text{ser}}\sin 40^\circ = 31.5\ \text{kN} \quad \text{(tension force per bolt)}$$

9.38.2.2 Design slip resistance

Preloading force:

$$F_{p,c} = 0.7 f_{ub} A_s = 0.7\cdot 80\cdot 2.45 = 137.2\ \text{kN}$$

Eq. 3.7

(For each M 20 tensile stress area $A_s = 2.45\ \text{cm}^2$)
The design slip resistance per bolt is:

$$F_{s,Rd,\text{ser}} = \frac{k_s n \mu (F_{p.c} - 0.8 F_{t,Ed,\text{ser}})}{\gamma_{M3}}$$

Eq. 3.8a

where

$$\gamma_{M3} = 1.25$$ Tab. 2.1

and therefore

$$F_{s,Rd,\text{ser}} = \frac{1.0 \cdot 1 \cdot 0.50 \cdot (137.2 - 0.8 \cdot 31.5)}{1.25} = 44.8kN > F_{v.Ed.\text{ser}} = 37.5 \text{ kN}$$

9.38.2.3 Shear resistance

Tab. 3.4

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 80 \cdot 3.14}{1.25} = 120.6 \text{ kN} > F_{v,Ed} = 54.4 \text{ kN}$$

9.38.2.4 Bearing resistance

Tab. 3.4

$$d_0 = 22 \text{ mm}$$
$$\alpha_b = \min\left\{\frac{140}{3 \cdot 22}, \frac{90}{3 \cdot 22} - \frac{1}{4}, \frac{80}{36}, 1.0\right\} = 1.0$$
$$k_1 = \min\left\{2.8\frac{35}{22} - 1.7, 1.4\frac{110}{22} - 1.7, 2.5\right\} = 2.50$$
$$F_{b,Rd} = \frac{2.5 \cdot 1.0 \cdot 36 \cdot 2.0 \cdot 1.8}{1.25} = 259.2 \text{ kN} > F_{v.Ed} = 54.4 \text{ kN}$$

9.38.2.5 Tension resistance of bolts

Tab. 3.4

$$F_{t,Rd} = \frac{0.9 \cdot 80 \cdot 2.45}{1.25} = 141.1 \text{ kN} > F_{t.Ed} = 45.6 \text{ kN}$$

9.38.2.6 Combined shear and tension

Tab. 3.4

It should valid:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 F_{t,Rd}} \leqslant 1$$
$$\frac{54.4}{120.6} + \frac{45.6}{1.4 \cdot 141.1} = 0.68 < 1$$

9.38.2.7 Punching shear resistance of plates

Tab. 3.4

As in Example 9.37 it is taken per each M 20 bolt:

$$B_{p.Rd} = 0.6 \cdot \pi \cdot 3.345 \cdot 1.836.0/1.25 = 326.9 \text{ kN}$$

($d_m = 3.345$ mm) and

$$F_{t.Ed} = 45.6 \text{ kN} < B_{p.Rd} = 326.9 \text{ kN}$$

Remark 20. The design checks required, depending on the category of the connection, are summarized in EN 1993-1-8. Table 3.2.

Remark 21. For the verification of the connection it was assumed that the flange of the column as well as the end plate of member CD are sufficiently stiff.

Remark 22. If the no sliping condition is not necessary, the use of Category B bolts (i.e. slip-resistant only at the serviceability limit state), leads to a more economic connection (regarding the number and the diameter of bolts).

9.39 Example: Bolted connection with a moment acting in its plane

In the continuous beam shown in Fig. 9.56 there is a pinned bolted connection at point A, with one side web plate. The cross-section of the beam is IPE 330. the steel grade S 235 and the design loads are $P_{Ed} = 97.9$ kN. The pinned connection is

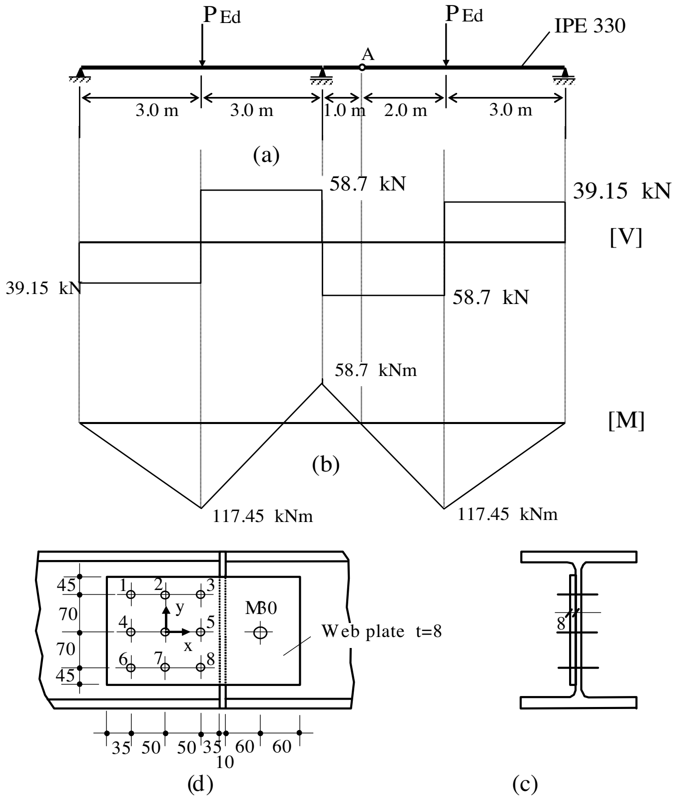


Fig. 9.56. Bolted beam-splice pinned connection

realized through a M 30 bolt, while M 12 bolts are used to connect the splice plate to the web of the beam. The threaded part of the bolts' shaft is out of the shear plane. Verify the capacity of the bolts for the following two cases:

1. Bolt class 5.6.
2. Bolt class 10.9.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.39.1 Actions

The shear forces and moments of the beam are presented in the diagrams shown in Fig. 9.56b.

The shear force $F_{Ed} = 58.7$ kN is applied at the M 30 bolt.

The shear force and the bending moment applied at the center of gravity of the 9M 12 bolt group are the following:

$$V_{Ed} = 58.7 \text{ kN}$$
$$M_{Ed} = 58.7 \cdot (6.0 + 1.0 + 3.5 + 5.0) = 58.7 \cdot 15.5 = 909.9 \text{ kNcm}$$

9.39.2 Class 5.6 bolts

9.39.2.1 Verification of M 30 bolt

Shear

$$F_{v,Rd} = 0.60 f_{ub} A/\gamma_{M2} = 0.60 \cdot \frac{1}{4} \cdot \pi \cdot 3.0^2 \cdot 50/1.25 = \\ = 169.6kN > F_{Ed} = 58.7 \text{ kN}$$ Tab. 3.4

Bearing resistance (one individual bolt)

$$F_{b,Rd} = 1.5 dt f_u/\gamma_{M2} = \\ = 1.5 \cdot 3.0 \cdot 0.75 \cdot 36/1.25 = 97.2 \text{ kN} > F_{Ed} = 58.7 \text{ kN}$$ Eq. 3.2

(the bolt should be provided with washers under both the head and the nut). 3.6.1(10)

9.39.2.2 Verification of M 12 bolts

Distribution of internal forces
The distribution of internal forces to the bolts might be assumed elastic (i.e. proportional to the distance from the center of rotation). The shear force V_{Ed} is assumed to be equally distributed amongst the nine bolts (provided that their size and class is the same). 3.12(1)

$$I_p = \sum(x^2 + y^2) = 6 \cdot 5.0^2 + 6 \cdot 7.0^2 = 444 \text{ cm}^2$$

Horizontal force due to moment at the corner bolts (No 1, 3, 6 and 8 bolts):

$$H_{x.Ed} = M_{Ed}y/I_p = 909.9 \cdot 7/444 = 14.35\ \text{kN}$$

Total vertical force at the most unfavorable bolt (No 3 and 8. Fig. 9.56c):

$$H_{y.Ed} = \frac{V_{Ed}}{n} + \frac{M_{Ed}}{I_p}x = \frac{58.7}{9} + \frac{909.9}{444} \cdot 5 = 6.52 + 10.25 = 16.77\ \text{kN}$$

Resultant force at the most unfavorable bolt (No 3 and 8):

$$H_{Ed} = \sqrt{H_{x,Ed}^2 + H_{y,Ed}^2} = \sqrt{(14.35^2 + 16.77^2)} = 22.07\ \text{kN}$$

Verification in shear

$$F_{v,Rd} = 0.60 \cdot 50 \cdot \frac{1}{4} \cdot \pi \cdot 1.2^2/1.25 = 27.14\ \text{kN} > F_{Ed} = 22.07\ \text{kN}$$ Tab. 3.4

Verification in bearing
The verification in bearing should be done separately for each of the two main directions y and x, with each of the components of the total force H_{Ed}. Tab. 3.4

Direction y
(holes with 13 mm diameter): Tab. 3.4

$$\alpha_b = \min\left\{\frac{45}{3 \cdot 13}, \frac{50}{36}, 1.0\right\} = 1.0$$

$$k_1 = \min\left\{2.8\frac{35}{13} - 1.7, 2.5\right\} = 2.5$$

$$F_{b,Rd.y} = k_1 \alpha_b f_u d t/\gamma_{M2} = \\ = 2.5 \cdot 3.6 \cdot 1.2 \cdot 0.6/1.25 = 51.84\ \text{kN} > H_{y.Ed} = 16.77\ \text{kN}$$

Direction x

$$\alpha_b = \frac{e_1}{3d_o} = \frac{35}{3 \cdot 13} = 0.897$$

$$k_1 = \min\left\{2.8\frac{45}{13} - 1.7, 2.5\right\} = 2.5 \quad \text{and}$$

$$F_{b,Rd.x} = 2.5 \cdot 0.897 \cdot 36 \cdot 1.2 \cdot 0.6/1.25 = 46.5\ \text{kN} > H_{x,Ed} = 14.35\ \text{kN}$$

Since the design shear resistance $F_{v,Rd}$ is less than the design bearing resistance $F_{b,Rd.}$ of the bolts, elastic linear distribution of internal forces is used, and there is not alternative possibility to use plastic analysis.

9.39.3 Class 10.9 bolts

9.39.3.1 Verification of M 30 bolt

The shear resistance for the class 10.9 is greater, while the bearing resistance remains the same as previously.

9.39.3.2 Verification of M 12 bolts

In this case the shear resistance per bolt is equal to:

$$F_{v,Rd} = 0.60 \cdot \frac{1}{4} \cdot \pi \cdot 1.2^2 \cdot 100/1.25 = 54.3 \text{ kN}$$

i.e. it is greater than the bearing resistance, which remains the same as in the previous calculations.

Due to this fact, as an alternative to the elastic analysis that used previously (paragraph 9.39.2), it is permitted to apply plastic analysis for the distribution of forces. 3.12(2)

In this case we will assume that at the ultimate limit state all the bolts are loaded by the same force N_i ($i = 1$ to 9), equal to their bearing resistance (46.5 kN). We will also assume that these forces act perpendicular to the lines r_i that connect each bolt to the instant center of rotation O (see Fig. 9.57). If the distance of this point from the center of gravity of the bolt group is x_0 the equilibrium equations will lead to the unknown values of V_{Rd} and x_0.

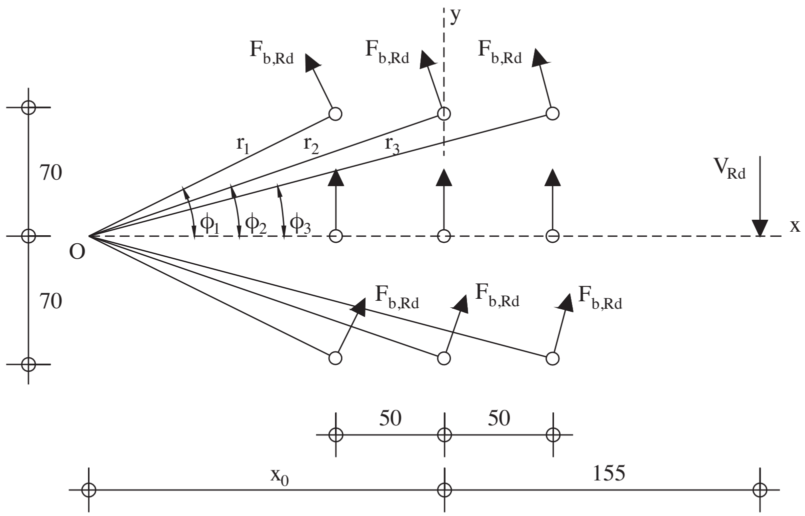


Fig. 9.57. Distribution of forces to the bolts in case of plastic analysis

Equilibrium of forces in the direction of y-y axis
(if $x_0 > 5$ cm)

$$V_{Rd} = 3F_{b,Rd} + 2F_{b,Rd}(\cos\phi_1 + \cos\phi_2 + \cos\phi_3)$$

where

$$\cos\phi_1 = (x_0 - 5)/r_1, \quad \cos\phi_2 = x_0/r_2, \quad \cos\phi_3 = (x_0 + 5)/r_3$$

and

$$r_1^2 = 7^2 + (x_0 - 5)^2, \qquad r_2^2 = 7^2 + x_0^2, \qquad r_3^2 = 7^2 + (x_0 + 5)^2$$

Equilibrium of moments

$$(15.5+x_0)V_{Rd} = F_{b,Rd}[2(r_1+r_2+r_3)+3x_0]$$

In case that the instant center of rotation (as happens in this example) is between the first and second vertical row of bolts (i.e. $x_0 < 5$ cm) the equilibrium equations are written as follows:

$$V_{Rd} = F_{b,Rd} + 2_{Fb,Rd}(\cos\phi_1 + \cos\phi_2 + \cos\phi_3)$$
$$(15.5+x_0)V_{Rd} = F_{b,Rd}[2(r_1+r_2+r_3)+10+x_0]$$

This system leads to

$$V_{Rd} = 159.8 \text{ kN} \quad \text{and} \quad x_0 = 4.3 \text{ cm}$$

and the connection is adequate since:

$$V_{Rd} = 159.8 \text{ kN} > V_{Ed} = 58.7 \text{ kN}$$

Remark 23. Every plastic distribution satisfying equilibrium equations is acceptable, provided that no excessive deformation of some holes' bearing surfaces is required for this distribution.

9.40 Example: Block shear tearing

The simply supported beam shown in Fig. 9.58 is connected through four M 20 bolts and appropriate web angles to a column at the left end and to a steel beam at the right end respectively. To realize the right-end connection a cope is necessary at the upper flange of the beam. Steel grade S 235. cross-section of the beam IPE 360. Calculate the maximum design shear resistance at both ends of the beam.

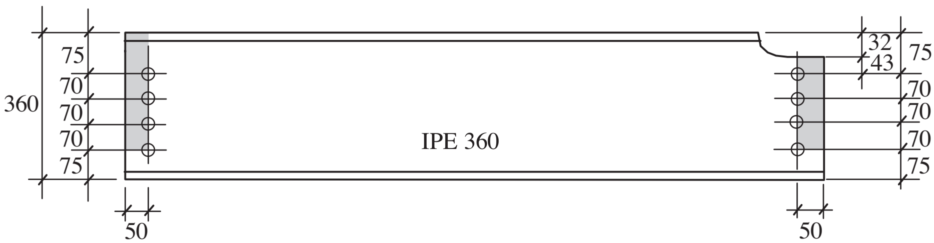


Fig. 9.58. Beam with appropriate preparation for the end connections

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.40.1 Design shear resistance of the cross-section

Shear area EN 1993-1-1. 6.2.6(3)

$$A_v = A - 2bt_f + (t_w + 2r)t_f = 72.7 - 2 \cdot 17 \cdot 1.27 + \\ + (0.8 + 2 \cdot 1.8) \cdot 1.27 = 35.11 cm^2 > \eta h_w t_w = 1 \cdot 36 \cdot 0.8 = 28.8 \text{ cm}^2$$

and

$$V_{pl,Rd} = \frac{A_v (f_y/\sqrt{3})}{2} = \frac{35.11 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 476.4 \text{ kN}$$

9.40.2 Design block tearing resistance at the left end of the beam

The probable block tearing will appear at the dashed area shown in Fig. 9.58.

Net tension area

$$A_{nt} = \left(50 - \frac{22}{2}\right) \cdot 8 = 312 \text{ mm}^2$$ 3.10.2

Net shear area

$$A_{nv} = (360 - 75 - 3.5 \cdot 22) \cdot 8 = 1664 \text{ mm}^2$$

and for eccentric loading:

$$V_{\text{eff},2.Rd} = \frac{0.5 f_u A_{nt}}{\gamma_{M2}} + \frac{f_y A_{nv}}{\sqrt{3}\gamma_{M0}} = \\ = \frac{0.5 \cdot 36.0 \cdot 3.12}{1.25} + \frac{23.5 \cdot 16.64}{\sqrt{3} \cdot 1.0} = 270.7 \text{ kN}$$ Eq. 3.10

Therefore, the maximum design shear force at the left end is:

$$V_{Rd} = \min(476.4, 270.7) = 270.7 \text{ kN}$$

9.40.3 Design block tearing resistance at the right end of the beam

Based on the block tearing resistance, this case is worse than the previous one (paragraph 9.40.2).

$$A_{nt} = 312 \text{ mm}^2$$

$$A_{nv} = (360 - 75 - 32 - 3.5 \cdot 22)8 = 1408 \text{ mm}^2$$

$$V_{\text{eff},2.Rd} = \frac{0.5 \cdot 36.0 \cdot 3.12}{1.25} + \frac{23.5 \cdot 14.08}{\sqrt{3} \cdot 1.0} = 236.0 \text{ kN}$$

and

$$V_{Rd} = \min(476.4, 236.0) = 236.0 \text{ kN}$$

9.41 Example: Simple beam-to-beam connection

Verify the connection shown in Fig. 9.59. in which the secondary beam IPE 270 transfers a design shear force $V_{Ed} = 60$ kN to the main beam IPE 360. Steel grade S 235. M 16 bolts of class 4.6.

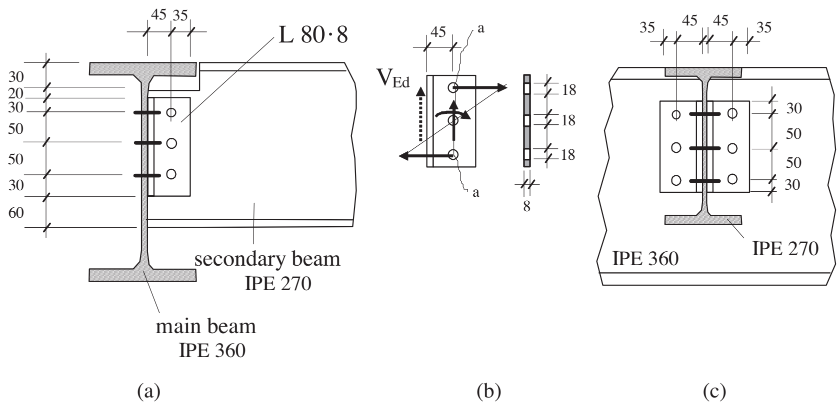


Fig. 9.59. Detail of a simple beam to beam connection

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.41.1 Distances and spacing of bolts

It is: $d_0 = 18$ mm and

$$e_1 = 30 \text{ mm}, \quad e_2 = 35 \text{ mm}, \quad p_1 = 50 \text{ mm}.$$ EN 1090. 8.2(2)

It is:

$$1.2d_0 = 1.2 \cdot 18 = 21.6 \text{ mm} < e_1 = 30 \text{ mm} < \\ < 4t + 40 \text{ mm} = 4 \cdot 6.6 + 40 = 66.4 \text{ mm}$$ Tab. 3.3

$$1.2d_0 = 1.2 \cdot 18 = 21.6 \text{ mm} < e_2 = 35 \text{ mm} < 66.4 \text{ mm}$$

$$2.2d_0 = 2.2 \cdot 18 = 39.6mm < p_1 = 50 \text{ mm} < \\ < \min(14t, 200 \text{ mm}) = \min(14 \cdot 6.6, 200 \text{ mm}) = 92.4 \text{ mm}$$ Tab. 3.3

9.41.2 Design shear forces of bolts

9.41.2.1 Bolts at the web of the main beam (single shear plane, 6M 16)

It is assumed that these bolts transfer only the design shear force V_{Ed} from the secondary to the main beam, without any simultaneous development of bending moment that additionally would cause tension forces to the bolts.

This assumption is closer to the real behavior of the connection, as the main beam becomes less distorsional.

Therefore, the design shear force that applies in each of the 6M 16 single shear plane bolts is equal to:

$$F_{V,Ed} = V_{\mathrm{E}d}/6 = 60/6 = 10\,\mathrm{kN}$$

9.41.2.2 Bolts at the web of the secondary beam (double shear plane, 3 M 16)

The shear force $V_{Ed} = 60$ kN, refering to the centroid of these three bolts, leads additionally to the following moment:

$$M_{Ed} = 60 \cdot 4.5 = 270\,\mathrm{kNcm}$$ (Fig. 9.59b)

which creates horizontal forces on the two external bolts:

$$F_{h,Ed} = \frac{M_{Ed}}{2p_1} = \frac{270}{2 \cdot 5} = 27\,\mathrm{kN}$$

In addition, the shear force is equally distributed to the three bolts:

$$F_{V,Ed} = V_{Ed}/3 = 60/3 = 20\,\mathrm{kN}$$

The resultant force of the external bolts is:

$$F_{Ed} = \sqrt{27^2 + 20^2} = 33.6\,\mathrm{kN}$$

9.41.3 Design resistance of bolts and verification

Tab. 3.4

9.41.3.1 Single shear plane bolts at the web of the main beam

Design shear resistance (the shear plane passes through the unthreaded part of the bolt):

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 40 \cdot \pi \cdot 1.6^2/4}{1.25} = 38.6\,\mathrm{kN}$$

Bearing resistance:

$$\alpha_b = \min\left(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1\right) = $$

$$= \min\left(\frac{30}{3 \cdot 18}, \frac{50}{3 \cdot 18} - \frac{1}{4}, \frac{40}{36}, 1\right) = 0.556$$

$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5\right\} =$$

$$= \min\left\{2.8\frac{35}{18} - 1.7, 1.4\frac{0}{18} - 1.7, 2.5\right\} = 2.5$$

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u d t}{\gamma_{M2}} = \frac{2.5 \cdot 0.556 \cdot 36 \cdot 1.6 \cdot 0.8}{1.25} = 51.2\,\mathrm{kN}$$

and

$$F_{v,Ed} = 10\,\mathrm{kN} < \min(F_{v,Rd}, F_{b,Rd}) = 38.6\,\mathrm{kN}$$

9.41.3.2 Double shear plane bolts at the web of the secondary beam

Design shear resistance (the shear plane passes through the unthreaded part of the bolt):

$$F_{v,Rd} = 2 \cdot 38.6 = 77.2\ \text{kN}$$

Bearing resistance:

$$F_{b,Rd} = \frac{2.5 \cdot 0.556 \cdot 36 \cdot 1.6 \cdot 0.66}{1.25} = 42.3\ \text{kN}$$

and

$$F_{Ed} = 33.6\ \text{kN} < \min(F_{v,Rd}, F_{b,Rd}) = 42.3\ \text{kN}$$

9.41.4 Check of angles at section a-a (Fig. 9.59b)

Shear force and moment per angle at section a-a:

$$V_{Ed} = 30\ \text{kN}$$
$$M_{Ed} = 135\ \text{kNcm}$$

Check if the hole existing in the tensioned part of the section a-a should be considered for the calculation of bending resistance. EN 1993-1-1. 6.2.5

The criterion is:

$$0.9\frac{A_{\text{net}}}{A} \geqslant \frac{f_y}{f_u}\frac{\gamma_{M2}}{\gamma_{M0}}$$ EN 1993-1-1. Eq. 6.16

where

$$A_{\text{net}} = 16 \cdot 0.8/2 - 1.5 \cdot 1.8 \cdot 0.8 = 4.24\ \text{cm}^2$$
$$A = 16 \cdot 0.8/2 = 6.4\ \text{cm}^2$$

Since:

$$0.9 \cdot \frac{4.24}{6.4} = 0.6 > \frac{23.5}{36} \cdot \frac{1.25}{1.0} = 0.82$$

the lower hole should be subtracted.

Therefore:

$$W_{pl} = \frac{0.8 \cdot 16^2}{4} - 1.8 \cdot 0.8 \cdot 5 = 44\ \text{cm}^3$$
$$W_{pl.Rd} = \frac{W_{pl} f_y}{\gamma_M} = \frac{44 \cdot 23.5}{1.0} = 1034\ \text{kNcm} > \text{M}_{\text{Ed}} = 135\ \text{kNcm}$$
$$V_{pl.Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{M0}} = \frac{12.8 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 173.7\ \text{kN} > V_{Ed} = 30\ \text{kN}$$

while

$$V_{Ed} = 30\ \text{kN} < \frac{1}{2} V_{pl,Rd} = \frac{173.7}{2} = 86.85\ \text{kN}$$

Therefore, the effect of shear force on the moment resistance may be neglected.

9.41.5 Design for block tearing of the secondary beam

3.10.2 + Fig. 3.8

For the bolt group of Fig. 9.60 subjected to eccentric loading the design block shear tearing resistance is given by:

$$V_{\text{eff},2.Rd} = 0.5 f_u A_{nt}/\gamma_{M2} + \frac{f_y A_{nv}}{\sqrt{3}\gamma_{M0}} \quad \text{Eq. 3.10}$$

where

$$A_{nt} = \left(4.5 - \frac{1.8}{2}\right) 0.66 = 2.38\ \text{cm}^2 \quad \text{(net area subjected to tension)}$$

$$A_{nv} = (15 - 2.5 \cdot 1.8) \cdot 0.66 = 6.93\ \text{cm}^2$$

Therefore,

$$V_{\text{eff},2.Rd} = \frac{0.5 \cdot 36.0 \cdot 2.38}{1.25} + \frac{23.5 \cdot 6.93}{\sqrt{3} \cdot 1.0} =$$
$$= 34.21 + 94.02 = 128.23\ \text{kN} > V_{Ed} = 60\ \text{kN}$$

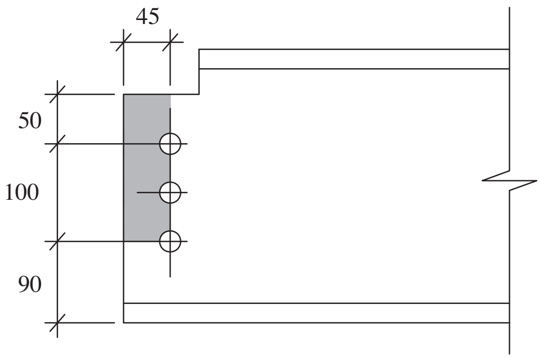


Fig. 9.60. Block tearing of secondary beam

9.42 Example: Pin connection

The pin connection of a member to a base plate is realized through two external plates with $a = 20$ mm thickness and a third intermediate plate $b = 30$ mm thick, which are connected by a pin of $d = 42$ mm diameter (Fig. 9.61a, b) and 10.9 class. The gap between the plates is $c = 1$ mm, the hole diameter $d_o = 45$ mm and the steel grade of the plates S 275. Verify the capacity of the pin to transfer a design force of $F_{Ed} = 400$ kN, if the corresponding design value of the force to be transferred under the characteristic load combination for serviceability limit state is $F_{Ed,ser} = 250$ kN.

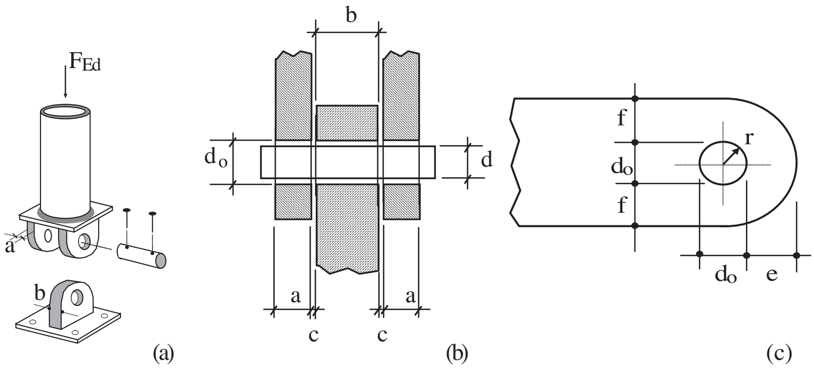


Fig. 9.61. Pin connection and geometry of pin ended members

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.42.1 Design internal forces and moments of the pin

Shear force perpendicular to the pin axis:

$$F_{Ed} = 400\ \text{kN}$$

Bending moment:

$$M_{Ed} = \frac{F_{Ed}}{8}(b + 4c + 2\alpha) = \frac{400}{8} \cdot (3.0 + 4 \cdot 0.1 + 2 \cdot 2.0) = \\ = 370\ \text{kNcm}$$

Fig. 3.11

9.42.2 Geometrical characteristics of pin ended plates (Fig. 9.61c)

Tab. 3.9

9.42.2.1 Intermediate plate (b = 30 mm)

It should be:

$$e_1 \geqslant \frac{F_{Ed}\gamma_{MO}}{2tf_y} + \frac{2d_o}{3} = \frac{400 \cdot 1.0}{2 \cdot 3.0 \cdot 27.5} + \frac{2 \cdot 4.5}{3} = \\ = 2.42 + 3.0 = 5.42\ \text{cm}$$

Tab. 3.9

It is selected $e_1 = 70$ mm, and

$$f_1 \geqslant \frac{F_{Ed}\gamma_{MO}}{2tf_y} + \frac{d_o}{3} = \frac{400 \cdot 1.0}{2 \cdot 3.0 \cdot 27.5} + \frac{4.5}{3} = \\ = 2.42 + 1.5 = 3.92\ \text{cm}$$

Tab. 3.9

It is selected $f_1 = 55$ mm.

9.42.2.2 External plates (a = 20 mm)

It should be:

$$e_2 \geqslant \frac{200 \cdot 1.0}{2 \cdot 2.0 \cdot 27.5} + \frac{2 \cdot 4.5}{3} = 1.82 + 3.0 = 4.82\ \text{cm}$$

It is selected $e_2 = 60$ mm, $\kappa\alpha\iota$

$$f_2 \geqslant \frac{200 \cdot 1.0}{2 \cdot 2.0 \cdot 27.5} + \frac{4.5}{3} = 1.82 + 1.5 = 3.32\ \text{cm}$$

It is selected $f_2 = 50$ mm.

9.42.3 Verification of capacity of the pin at the ultimate limit state

9.42.3.1 Shear resistance of the pin

It should be:

$$F_{v,Rd} = 0.60 A f_{up}/\gamma_{M2} \geqslant F_{V.Ed}$$

or

$$F_{v,Rd} = \cdot 0.60 \cdot \frac{1}{4} \cdot \pi \cdot 4.2^2 \cdot 100/1.25 = 665\ \text{kN} > F_{v.Ed} = \\ = \frac{1}{2} F_{Ed} = \frac{1}{2} 400 = 200\ \text{kN}$$

Tab. 3.10

9.42.3.2 Bending resistance of the pin

$$M_{Rd} = 1.5 W_{el} \quad f_{yp}/\gamma_{M0} = 1.5 \cdot \frac{1}{32} \cdot \pi \cdot 4.2^3 \cdot 90/1.0 = \\ = 982\ \text{kNcm} > M_{Ed} = 370\ \text{kNcm}$$

Tab. 3.10

9.42.3.3 Combined shear and bending resistance of the pin

It should be:

$$\left(\frac{M_{Ed}}{M_{Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leqslant 1$$

Tab. 3.10

or

$$\left(\frac{370}{982}\right)^2 + \left(\frac{200}{665}\right)^2 = 0.14 + 0.09 = 0.23 < 1$$

9.42.3.4 Bearing resistance of the plate

$$F_{b,Rd} = 1.5 d t f_y/\gamma_{MO} = \\ = 1.5 \cdot 4.2 \cdot 3.0 \cdot 27.5/1.0 = 520\ \text{kN} > \text{F}_{\text{b.Ed}} = 400\ \text{kN}$$

Tab. 3.10

The check refers to the intermediate plate with $b = 30$ mm which is the worst case, since the two external plates with $\alpha = 20$ mm should be checked only for the half load of 200 kN.

9.42.4 Verification of capacity of the pin at the serviceability limit state

9.42.4.1 Bending resistance

If the pin is intended to be replaceable, the following requirement should additionally be satisfied

$$M_{Rd,ser} = 0.8W_{el}f_{yp}/\gamma_{M6.ser} \geqslant M_{Ed,ser}$$ Tab. 3.10

or:

$$M_{Rd,ser} = 0.8\frac{1}{32}\pi \cdot 4.2^3 \cdot 90/1.0 = 523.7 \text{ kNcm} \geqslant$$
$$M_{Ed,ser} = \frac{F_{Ed,ser}}{8}(b+4c+2\alpha) = \frac{250}{8}(3+4\cdot 0.1+2\cdot 2.0) = 231.3 \text{ kNcm}$$

9.42.4.2 Bearing resistance

For a replaceable pin, the following requirement should also be satisfied

$$F_{b\cdot Rd\cdot ser} = 0.6dtf_y/\gamma_{M6.ser} \geqslant F_{b\cdot Ed,ser}$$ Tab. 3.10

or

$$F_{b\cdot Rd,ser} = 0.6\cdot 4.2\cdot 3.0\cdot 27.5/1.0 = 207.9 < F_{b\cdot Ed,ser} = 250 \text{ kN}$$

Therefore, the preconditions for a replaceable pin are not satisfied.

9.43 Example: Beam-splice connection

The continuity of the beam AB with IPE 400 cross-section and steel grade S 235 (Fig. 9.62a) is ensured at the point C, through flange and web plates, bolted in the right part of the beam through bolts of 4.6 class and welded to the left one (Fig. 9.62b, c, d). The beam is loaded by a concentrate design load $P_{Ed} = 121.5$ kN. The resistance verification for the following parts of the splice is required:

1. Beam cross-section.
2. M 20 bolts of flanges.
3. Fillet welds with 6 mm thickness of flange plates.
4. M 16 bolts of web.
5. Fillet welds with 4 mm thickness of web plates.
6. The bottom flange plate under tension.

The shear plane passes through the unthreaded portion of the bolts.
Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

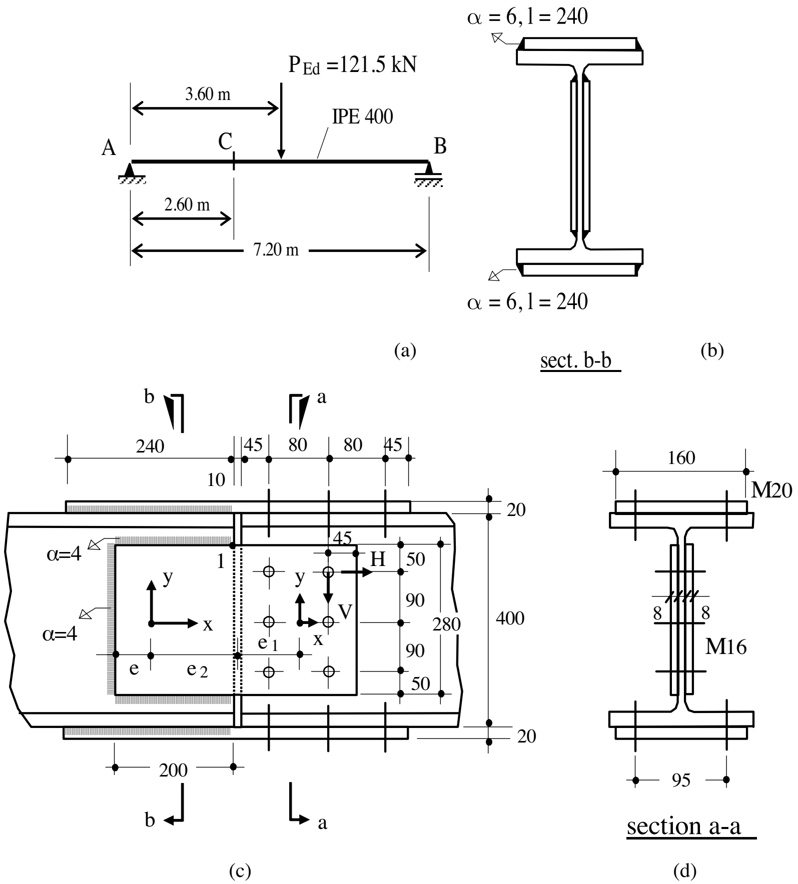


Fig. 9.62. Beam-splice connection

9.43.1 Beam cross-section check

Bending moment at midspan

$$M_{Ed} = \frac{1}{4} \cdot 121.5 \cdot 7.20 = 218.7 \text{ kNm}$$

Shear force

$$V_{Ed} = \frac{1}{2} \cdot 121.5 = 60.75 \text{ kN}$$

Design bending resistance (the cross-section is class 1)

$$W_{pl} = 1307 \text{ cm}^3$$

$$M_{c,Rd} = W_{pl} f_y / \gamma_{MO} = 1307 \cdot 23.5/(1.0 \cdot 100) = \\ = 307.1 \text{ kNm} > M_{Ed} = 218.7 \text{ kNm}$$

EN 1993-1-1. Eq. 6.13

Design shear resistance

$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{M0}$$ EN 1993-1-1. Eq. 6.18

where A_v is the shear area of the section:

$$\begin{aligned} A_v &= A - 2bt_f + (t_w + 2r)t_f = \\ &= 84.5 - 2 \cdot 18 \cdot 1.35 + (0.86 + 2 \cdot 2.1)1.35 = \\ &= 42.7\ \text{cm}^2 > \eta h_w t_w = 1.0 \cdot 37.3 \cdot 0.86 = 32.1\ \text{cm}^2 \end{aligned}$$ EN 1993-1-1. 6.2.6(3)

Therefore:

$$V_{pl,Rd} = 42.7 \cdot (23.5/\sqrt{3})/1.0 = 579.3\ \text{kN} > V_{Ed} = 60.75\ \text{kN}$$

Since

$$V_{Ed} = 60.75\text{kN} < \frac{1}{2}V_{pl,Rd} = \frac{1}{2}579.3 = 289.7\ \text{kN}$$

no reduction of the bending moment due to shear force at point C is necessary.

Verification of the reduced beam cross-section at splice location.

Design shear forces and moments at C.

$$M_{Ed,C} = 60.75 \cdot 2.60 = 157.95\ \text{kNm}$$
$$V_{Ed,C} = 60.75\ \text{kN}$$

Bending resistance

Flange area: $A_f = 18 \cdot 1.35 = 24.3\ \text{cm}^2$

Reduced flange area (holes Ø22):

$$A_{f,net} = 24.3 - 2 \cdot 2.2 \cdot 1.35 = 18.36\ \text{cm}^2$$

The following criterion

$$0.90\frac{A_{f,net}}{A_f} \geqslant \frac{f_y}{f_u}\frac{\gamma_{M2}}{\gamma_{M0}}$$ EN 1993-1-1. Eq. 6.16

leads to:

$$0.90 \cdot 18.36/24.3 = 0.68 < 23.5 \cdot 1.25/(36 \cdot 1.0) = 0.82$$

Since the criterion of paragraph 6.2.5 (4) is not satisfied, the cross-section reduction of the resistance due to the flange holes should be considered.

It is examined if an additional reduction of the beam resistance is required due to the web holes (holes Ø 18).

Cross-section area under tension:

$$A_t = \frac{1}{2} \cdot 84.5 = 42.25\ \text{cm}^2$$

Cross-section area under tension, reduced due to the holes:

$$A_{t,\text{net}} = 42.25 - 2 \cdot 2.2 \cdot 1.35 - 1.8 \cdot 0.86 = 34.76\ \text{cm}^2$$

The above-mentioned criterion is not satisfied again (bottom flange and part of the web under tension), and therefore the holes in the part of the web under tension should be considered for the calculation of the cross-section resistance. EN 1993-1-1. 6.2.5(5)

It is:

$$0.90 \cdot 34.76/42.25 = 0.741 < 23.5 \cdot 1.25/(36 \cdot 1.0) = 0.82$$

Reduced plastic section modulus

$$W_{pl,C} = 1307 - 2 \cdot 2.2 \cdot 1.35 \cdot \left(20 - \frac{1.35}{2}\right) - 1.8 \cdot 0.86 \cdot 9 = 1178\ \text{cm}^3$$

(the translation of the centroid axis is ignored)

$$M_{c,Rd,C} = 1178 \cdot 23.5/(1.0 \cdot 100) = 276.8\ \text{kNm} > M_{Ed,C} = 157.95\ \text{kNm}$$

Shear resistance

The elastic shear resistance is calculated EN 1993-1-1. 6.2.6

$$A_{w,\text{net}} = (40 - 2 \cdot 1.35 - 3 \cdot 1.8) \cdot 0.86 = 27.4\ \text{cm}^2$$

Since $A_{f,\text{net}}/A_{w,\text{net}} = 18.36/27.4 = 0.67 > 0.60$ EN 1993-1-1. Eq. 6.21

The effective shear stress on the web could be taken as

$$\tau_{Ed} = V_{Ed}/A_{w,\text{net}} = 60.75/27.4 = 2.22 kN/cm^2$$

and the check should be done through the relation

$$\frac{\tau_{Ed}}{f_y/(\sqrt{3}\gamma_{M0})} \leqslant 1 \quad \text{or} \quad \frac{2.22\sqrt{3}}{23.5 \cdot 1.0} = 0.17 < 1$$

EN 1993-1-1. Eq. 6.19

9.43.2 Check of M 20 bolts of flanges

The internal forces at splice location are distributed to the flange and web plates. The shear force is transfered entirely by the web plates. The bending moment is distributed in proportion to the stiffness of the plates. The condition for the above procedure is that the existing stiffness proportion between flanges and web of the beam is approximately kept the same between the flange and web splice plates.

$$I_w = 2 \cdot \frac{1}{12} \cdot 0.8 \cdot 28^3 = 2927\ \text{cm}^4$$

$$I_f = 2 \cdot 16 \cdot 2.0 \cdot 21.0^2 = 28224\ \text{cm}^4$$

$$M_w = 157.95 \cdot \frac{2927}{2927 + 28224} = 14.84\ \text{kNm}$$

$$M_f = 157.95 \cdot \frac{28224}{2927 + 28224} = 143.11\ \text{kNm}$$

Axial force of flange

$$N_{Ed} = \frac{M_f}{h} = 143.11/0.42 = 340.7\ \text{kN}$$

Shear resistance (six bolts)

$$F_{v,Rd} = 6 \cdot 0.6 \cdot 40 \cdot \frac{1}{4} \cdot \pi \cdot 2.0^2/1.25 = 361.9\ \text{kN} > N_{Ed} = 340.7\ \text{kN}$$ Tab. 3.4

Bearing resistance

$$\alpha_b = \min\left(\frac{45}{3 \cdot 22}, \frac{80}{3 \cdot 22} - \frac{1}{4}, \frac{40}{36}, 1\right) = 0.682$$

$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 2.5\right\} = \min\left\{2.8\frac{32.5}{22} - 1.7, 2.5\right\} = 2.43$$

and

$$F_{b,Rd} = 6 \cdot 2.43 \cdot 0.682 \cdot 36 \cdot 2.0 \cdot 1.35/1.25 = 773.2\ \text{kN} > N_{Ed} = 340.7\ \text{kN}$$ Tab. 3.4

9.43.3 Fillet welds with 6 mm thickness of flange plates

Design resistance of the weld (simplified method) Eq. 4.4

$$f_{v,wd} = f_u/(\sqrt{3}\beta_w \gamma_{Mw}) = 36/(\sqrt{3} \cdot 0.80 \cdot 1.25) = 20.7\ \text{kN/cm}^2$$ Tab. 4.1

Check:

$$\tau_{Ed} = 340.7/(2 \cdot 24 \cdot 0.6) = 11.8\ \text{kN/cm}^2 < f_{v,wd} = 20.7\ \text{kN/cm}^2$$

9.43.4 M 16 bolts of web

Determination of the force on the most unfavorable bolt (elastic analysis) 3.12(1)

$$I_p = \sum(x^2 + y^2) = 6 \cdot 4^2 + 4 \cdot 9^2 = 420\ \text{cm}^2$$

Moment that stresses the bolt group

$$M_{Ed} = M_w + V_{Ed} e_1 = 14.84 + 60.75 \cdot 0.09 = 20.31\ \text{kNm}$$

$$R_{Ed} = \left(H_{x.Ed}^2 + H_{y.Ed}^2\right)^{0.5} = \left[\left(\frac{M_{Ed} y}{I_p}\right)^2 + \left(\frac{V_{Ed}}{n} + \frac{M_{Ed} x}{I_p}\right)^2\right]^{0.5} =$$

$$= \left[\left(2031 \cdot \frac{9}{420}\right)^2 + \left(\frac{60.75}{6} + 2031 \cdot \frac{4}{420}\right)^2\right]^{0.5} =$$

$$= [43.5^2 + 29.5^2]^{0.5} = 52.5\ \text{kN}$$

Shear resistance of a bolt (two shear planes)

$$F_{v,Rd} = 2 \cdot 0.6 \cdot 40 \cdot \frac{1}{4} \cdot \pi \cdot 1.6^2/1.25 = 77.2\ \text{kN} > R_{Ed} = 52.5\ \text{kN}$$ Tab. 3.4

Bearing resistance

The check should be done for the most unfavorable direction x-x with the corresponding force $H_{x.Ed} = 43.5$ kN. It is: Tab. 3.4

$$\alpha_b = \min\left(\frac{45}{3\cdot 18}, \frac{80}{3\cdot 18} - \frac{1}{4}, \frac{40}{36}, 1\right) = 0.833$$
$$k_1 = \min\left\{2.8\frac{50}{18} - 1.7, 2.5\right\} = 2.5$$

and

$$F_{b,Rd} = 2.5 \cdot 0.833 \cdot 36 \cdot 1.6 \cdot 0.86/1.25 = 82.5\ \text{kN} > H_{x.Ed} = 43.5\ \text{kN}$$

9.43.5 Fillet welds with 4 mm thickness of web plates

Center of gravity of the welded connection (the distance e from the vertical weld is determined).

$$2 \cdot 20 \cdot 10 \cdot 0.4 = e \cdot (20 + 20 + 28) \cdot 0.4$$

and

$$e = 5.88\ \text{cm}$$

Second moment of area of the weld (in one side of the web) about the horizontal and vertical axes (x-x and y-y):

$$I_x = \frac{1}{12} \cdot 0.4 \cdot 28^3 + 2 \cdot 20 \cdot 14^2 \cdot 0.4 = 3868\ \text{cm}^4$$
$$I_y = 28 \cdot 0.4 \cdot 5.88^2 + 2 \cdot \frac{1}{12} \cdot 0.4 \cdot 20^3 + 2 \cdot 20 \cdot 0.4 \cdot (10 - 5.88)^2 = 1192\ \text{cm}^4$$

Moment that stresses the weld

$$M_{Ew} = M_w + V_{Ed} e_2 = 14.84 + 60.75 \cdot (20 - 5.88 + 0.5)/100 = 23.72\ \text{kNm}$$

Shear stresses at the most unfavorable point 1 for the welds on both sides (Fig. 9.62c)

$$\tau_{Edx} = 2372 \cdot 14/2 \cdot (3868 + 1192) = 3.28\ \text{kN/cm}^2$$
$$\tau_{Edy} = \frac{60.75}{[(20+20+28)\cdot 0.4]} + \frac{2372 \cdot 14.12}{2 \cdot (3868 + 1192)} = 5.54\ \text{kN/cm}^2$$

Resultant stress

$$\tau_{Ed} = (\tau_{Edx}^2 + \tau_{Edy}^2)^{0.5} = (3.28^2 + 5.54^2)^{0.5} = 6.44\ \text{kN/cm}^2 < f_{v,wd} = 20.7\ \text{kN/cm}^2$$

9.43.6 Bottom flange plate under tension

$$A = 16 \cdot 2.0 = 32\ \text{cm}^2$$
$$A_{\text{net}} = 32 - 2 \cdot 2.2 \cdot 2.0 = 23.2\ \text{cm}^2$$

Design tension resistance

$$N_{t,Rd} = \min\left(A f_y/\gamma_{M0}, 0.90 A_{\text{net}} f_u/\gamma_{M2}\right) =$$
$$= \min(32 \cdot 23.5/1.0, 0.90 \cdot 23.2 \cdot 36/1.25) =$$
$$= \min(752.0, 601.3\ \text{kN}) = 601.3\ \text{kN} > N_{Ed} = 340.7\ \text{kN}$$

EN 1993-1-1. 6.2.3

Remark 24. In this example, the design of the beam-splice connection was based on the internal forces at the splice location and not to the full capacity of the cross-section.

9.44 Example: Welded connection of two angles with a gusset plate

Determine the maximum design axial force N_{Ed} that may be transferred by the welded connection shown in Fig. 9.63. Steel grade S 235. Throat thickness of the fillet welds $a = 4$ mm.

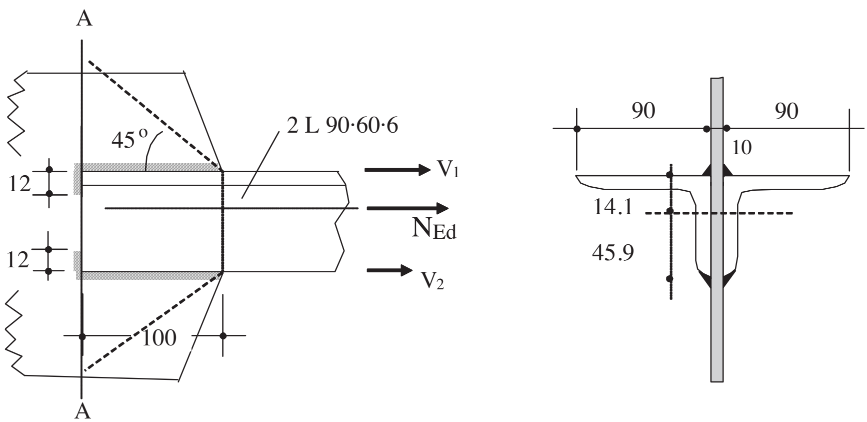


Fig. 9.63. Welded connection of two angles with a gusset plate

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.44.1 Resistance of the angles

Since the unequal leg angles are connected to the gusset plate with the smaller leg, when determining the design resistance of the cross section, the effective area should be taken as equal to the gross cross-sectional area of an equivalent equal leg angle having as leg size the one of the smaller leg. 4.13(3)

$$A = 2 \cdot 6.91 = 13.82\ \text{cm}^2 \qquad (2\text{L}60.6)$$
$$N_{Rd} = 13.82 \cdot 23.5/1.0 = 324.8\ \text{kN}$$

9.44.2 Resistance of the fillet weld

$$a = 4\ \text{mm} > 3\ \text{mm}$$ 4.5.2(2)

$$l = 100\ \text{mm} > 30\ \text{mm} \quad \text{or } 6a = 6 \cdot 4 = 24\ \text{mm}$$
$$= 100\ \text{mm} < 150a = 150 \cdot 4 = 600\ \text{mm}$$ 4.5.1(2)

and $\beta_{Lw} = 1$ (reduction factor due to probable excessive length of the weld). 4.11

The transversal welds of 12 mm length are not considered in the effective area, since 12 mm $<$ max(30 mm or $6a = 24$ mm) = 30 mm. 4.5.1(2)

The resistance of the weld is:

$$N_{w,Rd} = F_{w,Rd} \sum l = \frac{a f_u}{\sqrt{3} \beta_w \gamma_{M2}} \sum l =$$
$$= \frac{0.4 \cdot 36}{\sqrt{3} \cdot 0.8 \cdot 1.25} \cdot 4 \cdot 10 = 332.6\ \text{kN}$$ 4.5.3.3

where $\beta_w = 0.8$ is the correlation factor for S 235. Tab. 4.1

9.44.3 Resistance of the gusset plate

Assuming 45° angle of stress distribution into the gusset plate (Fig. 9.63), the resistance at section $A-A$ is: EN 1993-1-5. Fig. 3.4

$$N_{Rd} = A \frac{f_y}{\gamma_{M0}} = (2 \cdot 10 + 6) \cdot 1 \cdot \frac{23.5}{1.0} = 611\ \text{kN}$$

Therefore, the maximum design axial force is: $N_{Ed} = 324.8$ kN.

Remark 25. The eccentricity of the design load in respect to the welds is neglected in the above calculations. If it is considered, then the welds must transfer the following different forces (see Fig. 9.64):

$$V_1 = 324.8 \cdot 45.9/60 = 248.5\ \text{kN}$$
$$V_2 = 76.3\ \text{kN}$$

Since $V_1 = 248.5\ \text{kN} > N_{w,Rd}/2 = 332.6/2 = 166.3$ kN. The welds in this case are not adequate to resist the design load.

9.45 Example: Welded connection with an in-plane moment

Verify the capacity of the welded connection of a T bar with a gusset plate shown in Fig. 9.64. for a design force $S_d = 80$ kN. Steel grade S 355.

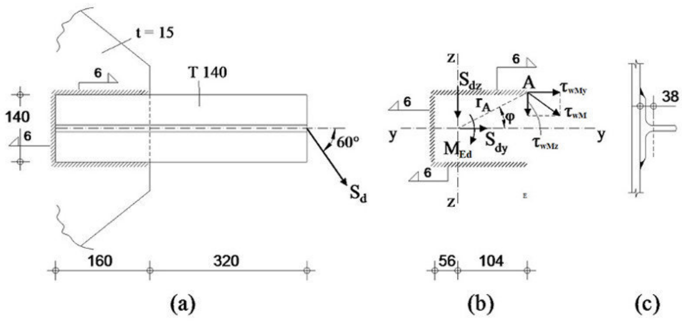


Fig. 9.64. Welded connection

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.45.1 Geometrical properties of the weld

Position of the center of gravity

$$y_0 = \frac{2 \cdot 6 \cdot 160 \cdot 80}{2 \cdot 160 \cdot 6 + 140 \cdot 6} = 55.65 \text{ mm} \approx 56 \text{ mm}$$

Second moment of area about the main axes

$$I_{wy} = 2 \cdot 0.6 \cdot 16 \cdot 7^2 + \frac{0.6 \cdot 14^3}{12} = 1078 \text{ cm}^4$$

$$I_{wz} = 0.6 \cdot 14 \cdot 5.6^2 + \frac{2 \cdot 0.6 \cdot (5.6^3 + 10.4^3)}{3} = 784 \text{ cm}^4$$

9.45.2 Actions refer to the center of gravity of the weld

$$S_{dz} = S_d \sin 60^\circ = 80 \cdot 0.866 = 69.3 \text{ kN}$$

$$S_{dy} = S_d \cos 60^\circ = 80 \cdot 0.500 = 40 \text{ kN}$$

$$M_{Ed} = S_{dz}(32 + 10.4) = 2937 \text{ kNcm}$$

The additional bending moment due to the eccentricity of the design force in refer to the plane of the weld is ignored (see the related remark at the end of the example).

9.45.3 Calculation of stresses

The stress distribution is performed using elastic analysis.

Most stressed point A.

$$r_A = \sqrt{10.4^2 + 7^2} = 12.54 \text{ cm}$$

$$\tau_{wM} = \frac{M_{Ed}}{I_{wy} + I_{wz}} r_A = \frac{2937}{1078 + 784} \cdot 12.54 = 19.78 \text{ kN/cm}^2$$

$$\tan\phi = \frac{70}{104} = 0.673, \quad \phi = 33.94^\circ$$

$$\cos\phi = 0.830$$

$$\sin\phi = 0.558$$

$$\tau_{wMz} = \tau_{wM}\cos\phi = 19.78 \cdot 0.830 = 16.42 \text{ kN/cm}^2$$

$$\tau_{wMy} = \tau_{wM}\sin\phi = 19.78 \cdot 0.558 = 11.04 \text{ kN/cm}^2$$

$$\tau_{wz} = \frac{S_{dz}}{A_s} = \frac{69.3}{0.6 \cdot (2 \cdot 16 + 14)} = 2.51 \text{ kN/cm}^2$$

$$\tau_{wy} = \frac{S_{dy}}{A_s} = \frac{40}{0.6 \cdot (2 \cdot 16 + 14)} = 1.45 \text{ kN/cm}^2$$

9.45.4 Check of stresses

$$\tau_{w,Ed} = \sqrt{(\tau_{wMy} + \tau_{wy})^2 + (\tau_{wMz} + \tau_{wz})^2} =$$

$$= \sqrt{(11.04 + 1.45)^2 + (16.42 + 2.51)^2} = 22.68 \text{ kN/cm}^2$$

Besides:

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\beta_w \gamma_{M2}} = \frac{51}{\sqrt{3} \cdot 0.9 \cdot 1.25} = 26.17 \text{ kN/cm}^2$$ Eq. 4.4

Therefore:

$$\tau_{w,Ed} = 22.68 \text{ kN/cm}^2 < f_{vw,d} = 26.17 \text{ kN/cm}^2$$

and the weld is sufficient.

9.45.5 Alternative approximate method to check the welds

The distribution of forces is performed according to Fig. 9.65. The moment is taken by the two parallel welds while the forces S_{dy}, S_{dz} are taken by the corresponding horizontal and vertical welds:

$z - z$ direction

$$\tau_{wz} = \frac{S_{dz}}{A_{vz}} = \frac{69.3}{0.6 \cdot 14} = 8.25 \text{ kN/cm}^2 < f_{w.Rd} = 26.17 \text{ kN/cm}^2$$

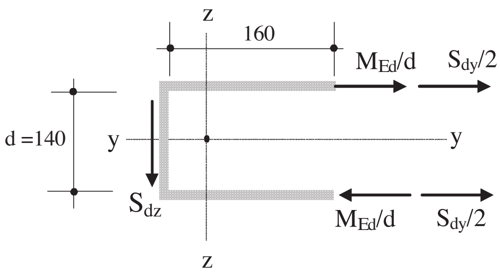


Fig. 9.65. Stress distribution using plastic analysis

$y-y$ direction

$$\max \tau_{wy} = \frac{M_{Ed}}{dA_{w,y}} + \frac{S_{dy}}{2A_{w,y}} = \left(\frac{2937}{14} + \frac{40}{2}\right) \cdot \frac{1}{0.6 \cdot 16} =$$
$$= 23.94 \text{ kN/cm}^2 < 26.17 \text{ kN/cm}^2$$

So, the welds are adequate by utilizing this approximate method too.

Remark 26. If the eccentricity of the design force S_d in respect to the plane of welds is considered (see Fig. 9.64c) additional bending moments about the $y-y$ and $z-z$ axes arise:

$$M_{yEd} = S_{dz}3.8 = 69.3 \cdot 3.8 = 263 \text{ kNcm}$$
$$M_{zEd} = S_{dy}3.8 = 40 \cdot 3.8 = 152 \text{ kNcm}$$

The corresponding direct stresses at point A are:

$$\sigma_{wMy} = \frac{M_{yEd}}{I_{wy}} \frac{14}{2} = \frac{263}{1078} \cdot \frac{14}{2} = 1.71 \text{ kN/cm}^2$$
$$\sigma_{wMz} = \frac{M_{zEd}}{I_{wz}} 10.4 = -\frac{152}{784} \cdot 10.4 = -2.02 \text{ kN/cm}^2$$

and the total equivalent stress:

$$\tau\prime_{w,Ed} = \sqrt{\tau^2_{w,eq} + (\sigma_{wMy} + \sigma_{wMz})^2} = \sqrt{22.68^2 + (1.71 - 2.02)^2} =$$
$$= 22.68 \text{ kN/cm}^2 < 26.17 \text{ kN/cm}^2$$

It maybe seen that the increase of the stress due to the eccentricity is negligible and is not considered in common applications.

9.46 Example: Welded bracket connection (short cantilever)

Verify the capacity of the welded bracket connection shown in Fig. 9.66. Design load $P_d = 20$ kN. Steel grade S 235.

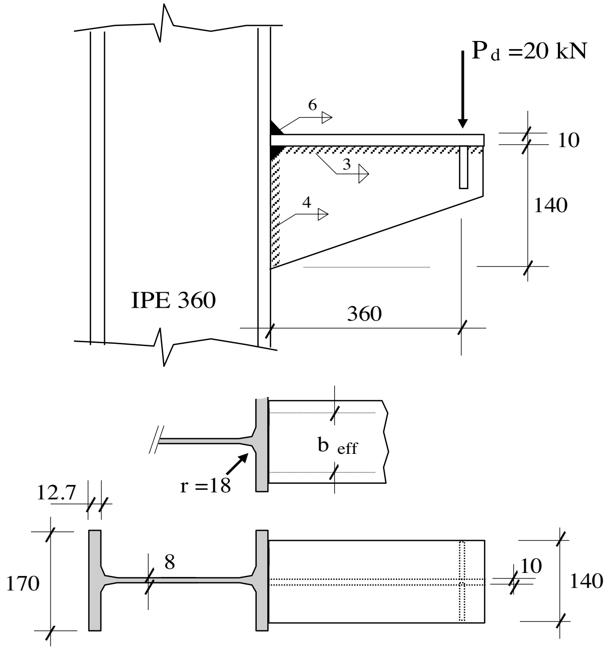


Fig. 9.66. Welded bracket connection

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.46.1 Actions at the bracket support

$$V_{Ed} = 20 \text{ kN}$$
$$M_{Ed} = 20 \cdot 0.36 = 7.2 \text{ kNm}$$

9.46.2 Effective width of the weld to the bracket flange

4.10

$$b_{\text{eff}} = t_w + 2s + 7kt_f = 8 + 2 \cdot 18 + 7 \cdot 1 \cdot 12.7 = 132.9 \text{ mm}$$ Eq. 4.6a

where

$$k = \frac{t_f}{t_p} \cdot \frac{f_{y,f}}{t_{y,p}} = \frac{12.7}{10} \cdot \frac{23.5}{23.5} = 1.27 \quad \text{but} \quad k < 1.0$$ Eq. 4.6b

and $s = r$ Eq. 4.6c

It is taken $k = 1.0$.

Since

$$b_{\mathrm{eff}} = 132.9 \text{ mm} > \left(\frac{f_{y,p}}{f_{u,p}}\right) \cdot b_p = \frac{23.5}{36} \cdot 140 = 91.4 \text{ mm}$$

no horizontal stiffeners are necessary for the I section of the column. 4.10(3)

Even if $b_{\mathrm{eff}} < b_p$, the welds connecting the flange of the bracket to the flange of the column shall be designed to transmit the design resistance to axial forces of the bracket flange $b_p t_p f_{y,p}/\gamma_{M0}$.

$$b_p t_p f_{y,p}/\gamma_{M0} = 14 \cdot 10 \cdot 23.5/1.0 = 329 \text{ kN}$$ 4.10(5)

and (bracket flange welds):

$$N_{w,Rd} = (14 + 2 \cdot 6.5) \cdot 0.6 \cdot 20.78 = 336.8 \text{ kN} > 329 \text{ kN}$$

where for the weld resistance:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = \frac{36/\sqrt{3}}{0.8 \cdot 1.25} = 20.78 \text{ kN/cm}^2$$

9.46.3 Geometrical properties of cantilever cross-section at the contact surface with the column's flange

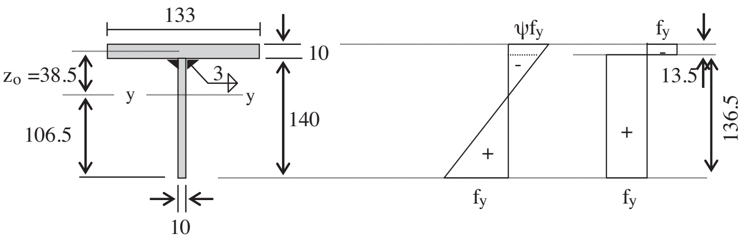


Fig. 9.67. Distribution of stresses in the cantilever

Elastic neutral axis (distance of the center of gravity from the flange's center)

$$z_o = \frac{14 \cdot 1 \cdot 7.5}{13.3 \cdot 1.0 + 14 \cdot 1.0} = 3.85 \text{ cm}$$

$$I_y = \frac{13.3 \cdot 4.35^3 - 12.3 \cdot 3.35^3}{3} + \frac{1 \cdot 10.65^3}{3} = 614 \text{ cm}^4$$

$$W_{el,y} = \frac{I_y}{z_u} = \frac{614}{10.65} = 57.7 cm^3$$

First moment of area:

$$S = 133 \cdot 10 \cdot 38.5 + 33.5 \cdot 10 \frac{33.5}{2} = 56816 \text{ mm}^3 = 56.8 \text{ cm}^3$$

Plastic neutral axis (distance from the upper fibre of flange)

$$z_o = 13.5 \text{ mm}$$

9.46.4 Cross-section classification (at support)

For elastic stress distribution (web):

$$\psi = -\frac{33.5 - 3 \cdot \sqrt{2}}{106.5} = -0.275 < 0$$ EN 1993-1-5. Tab. 4.2

$$k_\sigma = 0.57 - 0.21\psi + 0.07\psi^2 =$$
$$= 0.57 + 0.21 \cdot 0.275 + 0.07 \cdot 0.275^2 = 0.633$$

Therefore: EN 1993-1-1. Tab. 5.2

$$\frac{c}{t} = \frac{140 - 3 \cdot \sqrt{2}}{10} = 13.6 < 21\varepsilon\sqrt{k_\sigma} = 21 \cdot \sqrt{\frac{235}{235}} \cdot \sqrt{0.633} = 16.69$$

and the section of the web is at least class 3.

For plastic stress distribution, the web is entirely under compression:

$$\frac{c}{t} = 13.6 > 10\varepsilon = 10 \quad \text{and}$$
$$\frac{c}{t} = 13.6 < 14\varepsilon = 14$$

and the section of the web is also of class 3.

The flange is not examined since is under tension.

$$\tau_{Ed} = \frac{V_{Ed}S}{I \cdot t} = \frac{20 \cdot 56.8}{614 \cdot 1} = 1.85\ \text{kN/cm}^2$$

9.46.5 Verification of resistance of the section at support

$$M_{c.Rd} = W_{el} f_y / \gamma_{M0} = 57.7 \cdot 23.5/1.0 =$$
$$= 1356\ \text{kNcm} = 13.56\ \text{kNm}$$ EN 1993-1-1. Eq. 6.14

$$M_{Ed} = 7.2\ \text{kNm} < M_{c,Rd} = 13.56\ \text{kNm}$$ EN 1993-1-1. Eq. 6.12

Maximum shear stress:

$$\tau_{Ed} = \frac{V_{Ed}S}{It} = \frac{20 \cdot 56.8}{614 \cdot 1} = 1.85\ \text{kN/cm}^2$$ EN 1993-1-1. Eq. 6.20

and:

$$\tau_{Ed} = 1.85\ \text{kN/cm}^2 < \frac{f_y}{\sqrt{3}\gamma_{M0}} = \frac{23.5}{\sqrt{3} \cdot 1.0} =$$
$$= 13.57\ \text{kN/cm}^2$$ EN 1993-1-1. Eq. 6.19

Moreover, since:

$$\tau_{Ed} = 1.85\ \text{kN/cm}^2 < 0.5\frac{f_y}{\sqrt{3}\gamma_{M0}} = 0.5 \cdot \frac{23.5}{\sqrt{3} \cdot 1.0} = 6.78\ \text{kN/cm}^2$$

so, reduction of the moment is not required.

In addition, shear buckling resistance of the web is not required since:

$$\frac{h_w}{t_w} = \frac{140}{10} = 14 < 72\frac{\varepsilon}{\eta} = 72\frac{1.0}{1.0} = 72$$ EN 1993-1-1. Eq. 6.22

The cross-section of the cantilever is sufficient. EN 1993-1-1. 5.4.7(2)

9.46.6 Check of the weld between column flange and of the bracket web

$a = 3$ mm,

$$\tau_{Ed} = \frac{V_{Ed}S_y}{I \cdot 2a} = \frac{20 \cdot 13.3 \cdot 1.0 \cdot 3.85}{614 \cdot 2 \cdot 0.3} = 2.78\ \mathrm{kN/cm^2} < 20.79\ \mathrm{kN/cm^2}$$

and the weld is adequate.

9.46.7 Bracket to column welding

a) Geometrical and stiffness properties of the weld (elastic stress distribution)

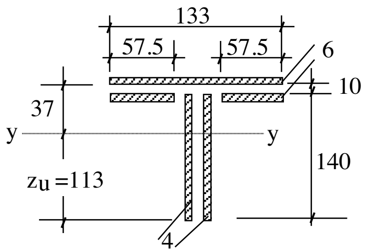


Fig. 9.68. Cross-section of the weld

$$A_{wf} = 133 \cdot 6 + 2 \cdot (57.5 \cdot 6) = 1488\ \mathrm{mm^2}$$
$$A_{ww} = 2 \cdot 140 \cdot 4 = 1120\ \mathrm{mm^2}$$

Center of gravity (distance from the lower fibre of the web)

$$z_u = \frac{133 \cdot 6 \cdot 150 + 2 \cdot 57.5 \cdot 6 \cdot 140 + 2 \cdot 140 \cdot 4 \cdot 70}{1488 + 1120} = 113\ \mathrm{mm}$$

$$I_{w,y} = 13.3 \cdot 0.6 \cdot 3.7^2 + 2 \cdot 5.75 \cdot 0.6 \cdot 2.7^2 + \frac{0.4 \cdot \left(2.7^3 + 11.3^3\right)}{3} = 354.6\ \mathrm{cm^4}$$

$$W_{w.el,y} = \frac{354.6}{11.3} = 31.4\ \mathrm{cm^3}$$

b) Check of stresses
The check will be performed at the lower fibre. 4.5.3(3)

$$\tau_{//} = \tau_{Ed} = \frac{V_{Ed}}{A_{ww}} = \frac{20}{11,2} = 1,786\ \mathrm{kN/cm^2}$$
$$\sigma_{\perp} = \sigma_{Ed} = \frac{M_{Ed}}{W_{w,el,y}} = \frac{7.2 \cdot 100}{31.4} = 22.93\ \mathrm{kN/cm^2}$$

and equivalent normal stress:

$$\sigma_{w,Ed} = \sqrt{3 \cdot \tau_{11}^2 + \sigma_{\perp}^2} = \sqrt{3 \cdot 1.786^2 + 22.93^2} = 23.14\ \mathrm{kN/cm^2}$$

In addition:

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\gamma_{M2}\beta_w} = \frac{36}{\sqrt{3} \cdot 1.25 \cdot 0.8} = 20.79\ \mathrm{kN/cm^2},$$ Eq. 4.4

$$\sigma_{w,Ed} = 23.14\ \mathrm{kN/cm^2} > f_{vw.d} = 20.79\ \mathrm{kN/cm^2}$$

and the weld is not sufficient.

To resolve the issue following choices may be taken: (a) change the steel grade of both cantilever's plates and column, (b) increase the thickness of the vertical welds, or (c) increase the height of cantilever. It is chosen here to increase the thickness of the vertical welds to $a = 5$ mm.

Thus:

$$\frac{c}{t} = \frac{140 - 5 \cdot \sqrt{2}}{10} = 13,29 > 10.78$$
$$< 51.44$$

and the web is class 3.

Moreover:

$$A_{ww} = 1400\ \mathrm{mm^2} = 14\ \mathrm{cm^2}$$
$$I_{w,y} = 421.3\ \mathrm{cm^4}$$
$$W_{w.el,y} = 38.6\ \mathrm{cm^3}$$
$$\tau_{Ed} = 1.429\ \mathrm{kN/cm^2}$$
$$\sigma_{Ed} = 18.65\ \mathrm{kN/cm^{2\cdot}} \qquad \text{and}$$
$$\sigma_{w,Ed} = 18.71\ \mathrm{kN/cm^2} < \mathrm{f_{vw.d}} = 20.79\ \mathrm{kN/cm^2}$$

Therefore, the weld of the web for $a = 5$ mm is sufficient.

9.46.8 Alternative check of the weld

In the previous paragraphs, the welding verification was performed using the simplified method (see 5.4.3.1).

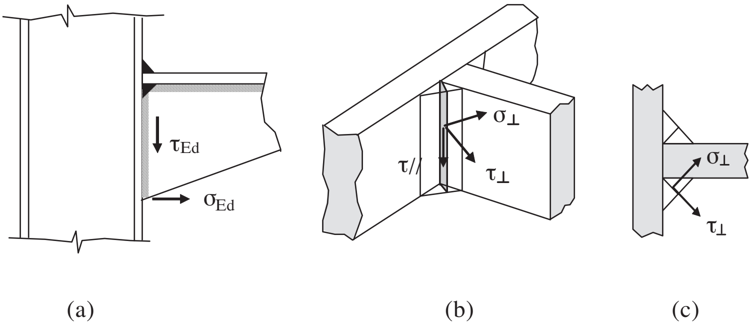


Fig. 9.69. Stresses at the weld according to Fig. 4.5 of EN 1993-1-8

As an example, the verification is repeated, using the directional method. 4.5.3.2

$$\tau_{//} = \tau_{Ed} = 1.43\,\text{kN/cm}^2 \sigma_{\perp} = \sigma_{Ed} \sin 45^\circ = 18.65 \cdot 0.707 = 13.19\,\text{kN/cm}^2$$
$$\tau_{\perp} = \sigma_{Ed} \cos 45^\circ = 18.65 \cdot 0.707 = 13.19\,\text{kN/cm}^2$$

$$\sigma_{Ed} = \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{//}^2)} = \sqrt{13.19^2 + 3 \cdot (13.19^2 + 1.43^2)} = \\ = 26.50\,\text{kN/cm}^2$$ Eq. 4.1

Therefore:

$$\sigma_{Ed} = 26.50\,\text{kN/cm}^2 < \frac{\text{f}_\text{u}}{\beta_\text{w} \gamma_{\text{M2}}} = \frac{36}{0.8 \cdot 1.25} = 36\,\text{kN/cm}^2$$
$$\sigma_{\perp} = 13.19\,\text{kN/cm}^2 < \frac{f_u}{\gamma_{M2}} = \frac{36}{1.25} = 28.8\,\text{kN/cm}^2$$

and the weld is sufficient.

9.47 Example: Welded short cantilever under combined stresses

The short cantilever shown in Fig. 9.70 made by a $140 \cdot 80$ RHS, is connected to a column through an all-around fillet weld with 6 mm thickness. The cantilever is loaded eccentrically at the free end by a design load $S_d = 100$ kN. The weld's verification is required. Steel grade S 235.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

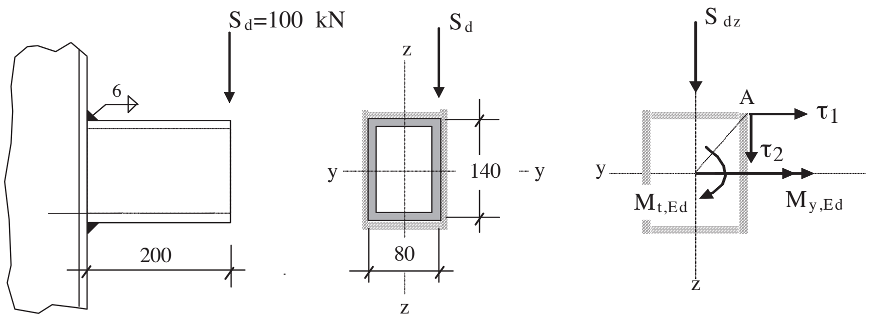


Fig. 9.70. Welded short cantilever

9.47.1 Geometrical properties of the fillet weld

$$A = 2 \cdot 0.6 \cdot (14+8) = 26.4 \text{ cm}^2$$
$$I_y = 2 \cdot 8 \cdot 0.6 \cdot 7^2 + 2 \cdot \frac{0.6 \cdot 14^3}{12} = 744.8 \text{ cm}^4$$
$$I_z = 2 \cdot 14 \cdot 0.6 \cdot 4^2 + 2 \cdot \frac{0.6 \cdot 8^3}{12} = 320 \text{ cm}^4$$
$$I_p = I_y + I_z = 744.8 + 320 = 1064.8 \text{ cm}^4$$

9.47.2 Actions at the center of gravity of the weld

$$S_{dz} = S_d = 100 \text{ kN}$$
$$M_{y,Ed} = S_d \cdot 20 = 100 \cdot 20 = 2000 \text{ kNcm}$$
$$M_{t,Ed} = S_d \cdot 4 = 100 \cdot 4 = 400 \text{ kNcm}$$

9.47.3 Calculation of stresses

Point A (most unfavorable):

$$\sigma_A = \frac{M_{y,Ed}}{I_y}\frac{14}{2} = \frac{2000}{744.8} \cdot \frac{14}{2} = 18.80 \text{ kN/cm}^2$$
$$\tau_1 = \frac{M_{t,Ed}}{I_p}\frac{14}{2} = \frac{400}{1064.8} \cdot \frac{14}{2} = 2.63 \text{ kN/cm}^2$$
$$\tau_2 = \frac{M_{t,Ed}}{I_p}\frac{8}{2} + \frac{S_{dz}}{A} = \frac{400}{1064.8} \cdot \frac{8}{2} + \frac{100}{26.4} = 1.50 + 3.79 = 5.29 \text{ kN/cm}^2$$

Resultant stress:

$$\sigma_{Ed} = \sqrt{\sigma_A^2 + \tau_1^2 + \tau_2^2} = \sqrt{18.8^2 + 2.63^2 + 5.29^2} = 19.71 \text{ kN/cm}^2$$

9.47.4 Check of the weld

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\beta_w \gamma_{M2}} = \frac{36}{\sqrt{3}\cdot 0.8\cdot 1.25} = 20.78\ \mathrm{kN/cm^2}$$ 4.5.3.3

and

$$\sigma_{Ed} = 19.71\ \mathrm{kN/cm^2} < f_{vw,d} = 20.78\ \mathrm{kN/cm^2}$$

Therefore, the weld is sufficient.

9.47.5 Alternative check of the weld, using directional method 4.5.3.2

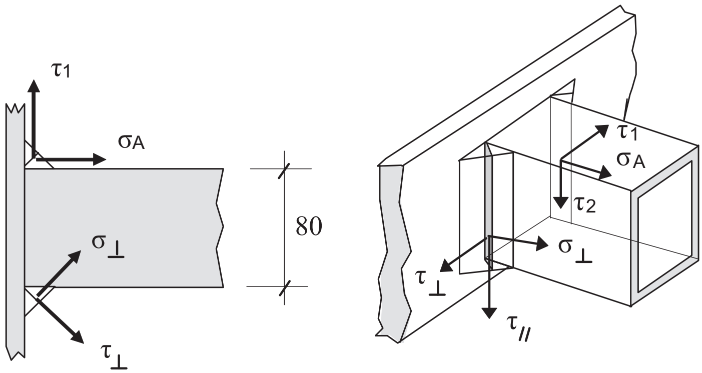


Fig. 9.71. Alternative check of the welds

$$\tau_{//} = \tau_2 = 5.29\ \mathrm{kN/cm^2}$$
$$\sigma_{\perp} = \sigma_A \sin 45^\circ + \tau_1 \cos 45^\circ =$$
$$= (18.80 - 2.63)\cdot 0.707 = 11.43\ \mathrm{kN/cm^2}$$
$$\tau_{\perp} = (18.80 + 2.63)\cdot 0.707 = 15.15\ \mathrm{kN/cm^2}$$

and

$$\sigma_{Ed} = \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{//}^2)} = \sqrt{11.43^2 + 3\cdot(15.15^2 + 5.29^2)} = 30.05\ \mathrm{kN/cm^2}$$

Therefore:

$$\sigma_{Ed} = 30.05\ \mathrm{kN/cm^2} < \frac{f_u}{\beta_w \gamma_{Mw}} = \frac{36}{0.8\cdot 1.25} = 36\ \mathrm{kN/cm^2} \quad \text{and}$$
$$\sigma_{\perp} = 11.43\ \mathrm{kN/cm^2} < \frac{f_u}{\gamma_{Mw}} = \frac{36}{1.25} = 28.8\ \mathrm{kN/cm^2}$$

and the weld is sufficient.

9.48 Example: Intermittent fillet welds in a plate girder

The simply supported plate girder shown in Fig. 9.72 is laterally restrained. The connection of the plates is obtained through intermittent fillet welds of 3 mm thickness, while the beam is loaded by the design forces shown in the same Figure. Steel grade S 235. Verify the capacity of the plate girder cross-section and of the intermittent welds connecting the web and the flanges. Determine the required width b of the additional plates with 8 mm thickness (Fig. 9.72d), which are connected to the flanges with intermittent welds of length $lw_2 = 80$ mm, so that the new cross-section could resist an 75% increased design loading.

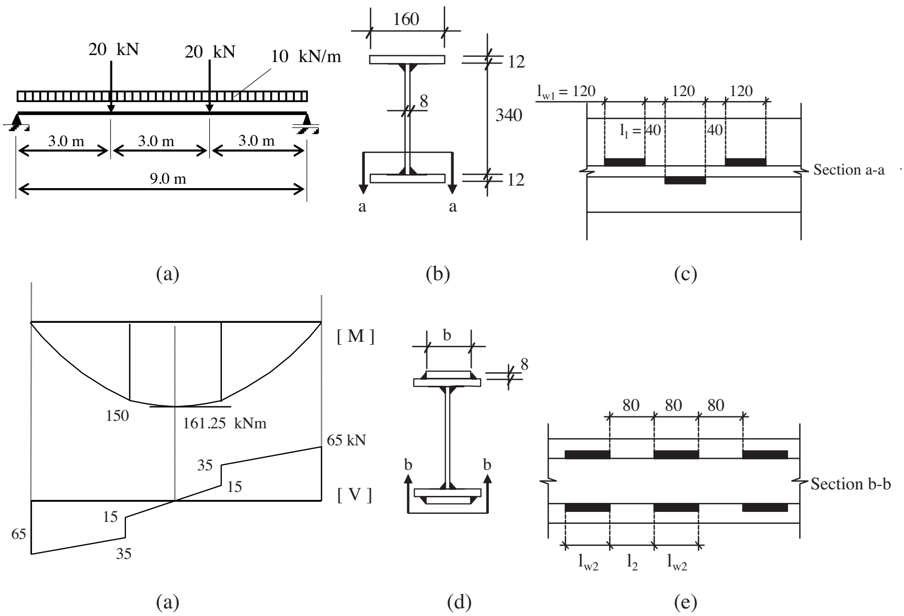


Fig. 9.72. Simply supported plate girder with intermittent fillet welds

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.48.1 Calculation of actions

In Fig. 9.72a the diagrams of bending moments and shear forces for the design loads are presented.

9.48.2 Resistance of the cross-section shown in Fig. 9.72b

9.48.2.1 Cross-section classification

EN 1993-1-1. Tab. 5.2

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/235} = 1$$

Web:

$$\frac{c}{t} = \frac{340 - 2 \cdot 3 \cdot \sqrt{2}}{8} = 41.4 < 72\varepsilon = 72$$

The web is class 1
Flange:

$$c = \frac{(160 - 8)}{2} - 3 \cdot \sqrt{2} = 71.8 \text{ mm} \frac{\text{c}}{\text{t}} \quad = \frac{71,8}{12} = 5.98 < 9\varepsilon = 9$$

The flange is class 1, and therefore the whole cross-section is class 1.

9.48.2.2 Resistance in bending of the section

$$W_{pl} = 2 \cdot \left(12 \cdot 16.0 \cdot 17.6 + 0.8 \cdot 17.0 \cdot \frac{17.0}{2}\right) = 907 \text{ cm}^3$$

$$M_{pl,Rd} = W_{pl} f_y / \gamma_{Mo} = 907 \cdot 23.5/1.0 = \\ = 21314 \text{ kNcm} = 213.1 \text{ kNm}$$

EN 1993-1-1. Eq. 6.13

and

$$M_{Ed} \leqslant M_{c.Rd}$$

or

$$M_{Ed} = 161.25 < M_{c.Rd} = M_{pl.Rd} = 213.1 \text{ kNm}$$

EN 1993-1-1. Eq. 6.12

9.48.2.3 Check of shear force

$$V_{pl,Rd} = \frac{A_v f_y}{\sqrt{3}\gamma_{M0}} = \frac{34.0 \cdot 0.8 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 369 \text{ kN} > 65 \text{ kN}$$

EN 1993-1-1. Eq. 6.18

Moreover:

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{65}{369} = 0.18 < 0.50$$

Thus, it is not necessary to reduce the bending moment resistance due to shear forces. In any case, the maximum value of the shear forces does not coexist with the maximum value of the bending moments.

EN 1993-1-1. 6.2.10(2)

9.48.3 Check of capacity of the intermittent welds

The maximum value of the shear flow at the interface between web and flange is equal to:

$$T'_{Ed} = \frac{V_{Ed} \cdot S'_y}{I_y} = \frac{65 \cdot 1.2 \cdot 16 \cdot 17.6}{14520} = 1.513 \text{ kN/cm}$$

EN 1993-1-1. Eq. 6.20

where:

$$I_y = \frac{16 \cdot 36.4^3 - 15.2 \cdot 34^3}{12} = 14520 \text{ cm}^4$$

The design value of the shear flow (weld force per unit length) for each intermittent weld is:

$$T_{Ed} = T'_{Ed}(4+12) = 24.2\,\text{kN}.$$ Fig. 4.7

The design weld resistance per unit length of the intermittent weld according to simplified method is: 4.5.3.3

$$F_{w.Rd} = f_{v.wd} \cdot a \cdot l_w = \frac{f_u a l_w}{\beta_w \gamma_{M2}\sqrt{3}} = \frac{36 \cdot 0.3 \cdot 12}{0.8 \cdot 1.25 \cdot \sqrt{3}} = 74.8\,\text{kN}$$ Eq. 4.3

where

$$l_w = 12\,\text{cm} \quad \text{(length of intermittent weld)}$$ Eq. 4.4

$$\beta_w = 0.8 \quad \text{for} \quad S235$$ Tab. 4.1

and

$$T_{Ed} = 24.2 < F_{w,Rd} = 74.8\,\text{kN}$$

According to the restrictions included in EN 1993-1-8 regarding the intermittent welds, reduction of the weld thickness $a = 3$ mm or of the weld length l_{w1} or increase of the free distance $l_1 = 40$ mm (Fig. 9.72c) are not permitted as: 4.5.2(2)
For the length l_{w1} it should be:

$$l_{w1} \geqslant 0.75b = 0.75 \cdot 160 = 120\,\text{mm}$$ Fig. 4.1

while for the free distance l_1 based on the compressed flange of the beam, it should be:

$$l_1 \leqslant \min(12t, 12t_1, 0.25b, 200\,\text{mm}) = \min(12 \cdot 12, 12 \cdot 8, 0.25 \cdot 160, 200\,\text{mm}) = 40\,\text{mm}$$

9.48.4 Determination of the width b of the additional plate

The maximum actions due to the 75% increase of the design forces are:

$$\max M_{Ed} = 1.75 \cdot 161.25 = 282.2\,\text{kNm}$$
$$\max V_{Ed} = 1.75 \cdot 65 = 113.8\,\text{kN}$$
$$W_{pl} = 907 + 2 \cdot 0.8 \cdot b \cdot 18.6 = 907 + 29.76 \cdot b\,\text{cm}^3$$
$$M_{pl,Rd} = W_{pl} \cdot f_y/\gamma_{M0} = (907 + 29.76 \cdot b) \cdot 23.5/1.0$$

The relation:

$$M_{Ed} \leqslant M_{pl,Rd}$$

leads to $b \geqslant 9.87$ cm. A width of $b = 100$ mm is chosen. Thus

$$M_{pl,Rd} = (907 + 29.76 \cdot 10) \cdot 23.5/1.0 = 283.1\,\text{kNm} \geqslant M_{Ed} = 282.2\,\text{kNm} \approx \text{M}_{\text{Ed}}$$

and

$$V_{Ed} = 113.8\,\text{kN} < V_{pl,Rd} = 369\,\text{kN}.$$

9.48.5 Check of capacity of the length l_{w2} of the intermittent welds

The new second moment of area is equal to:

$$I_y = 14520 + 2 \cdot 0.8 \cdot 10 \cdot 18.6^2 = 20055\ \text{cm}^4$$

therefore:

$$T'_{Ed} = \frac{V_{Ed} S_y}{I_y} = \frac{113.8 \cdot (0.8 \cdot 10 \cdot 18.6)}{20055} = 0.84\ \text{kN/cm}$$

Fig. 4.7

and

$$T_{Ed} = 0.84(l_2 + l_{w2}) = 0.84(8 + 8) = 13.4\ \text{kN}$$

The resistance of each pair of this intermittent weld is:

$$F_{w,Rd} = \frac{0.3 \cdot 36\,(2 l w_2)}{\sqrt{3} \cdot 0.8 \cdot 1.25} = 99.7\ \text{kN}$$

Eq. 4.3 and Eq. 4.4

($l_{w2} = 8$ cm) and finally:

$$T_{Ed} = 13.4\ \text{kN} < F_{w.Rd} = 99.7\ \text{kN}$$

Regarding the corresponding intermittent weld between web and flange:

$$S_y = 0.8 \cdot 10 \cdot 18.6 + 1.2 \cdot 16 \cdot 17.6 = 486.7\ \text{cm}^3$$
$$T'_{Ed} = \frac{V_{Ed} S_y}{I_y} = \frac{113.8 \cdot 486.7}{20055} = 2.76\ \text{kN/cm}$$
$$T_{Ed} = 2.76 \cdot (12 + 4) = 44.2\ \text{kN} < \mathrm{F_{w \cdot Rd}} = 74.8\ \text{kN}$$

9.49 Example: Beam to column bolted connection

The maximum bending moment and the corresponding shear force of the bolted connection between an IPE 330 beam and a HEB 320 column, shown in Fig. 9.73 are to be determined. In addition, the rotational stiffness of the joint for a design moment $M_{Ed} = 100$ kNm is required. M 20/8.8 bolts are used (the shear plane passes through the threaded part of the bolt). Steel grade S 235 (the verification of the welded connection between the end plate and the beam is not examined in this example).

Cross-sections' data:

Beam: IPE330: $b_{fb} = 160$ mm, $r_b = 18$ mm, $t_{fb} = 11.5$ mm, $t_{wb} = 7.5$ mm, $h_b = 330$ mm.

Column: HEB320: $b_{fc} = 300$ mm, $t_{fc} = 20.5$ mm, $t_{wc} = 11.5$ mm, $r_c = 27$ mm, $h_c = 320$ mm.

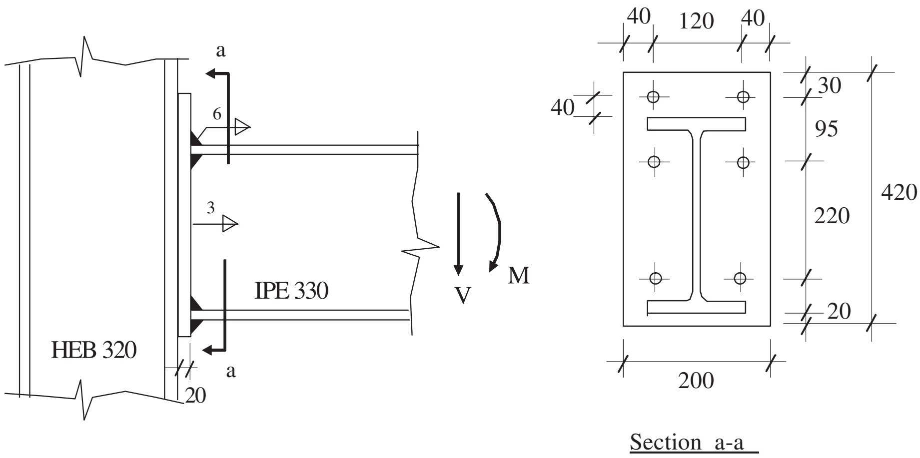


Fig. 9.73. Beam to column bolted connection

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

Introduction

The procedure to determine the design bending moment of the bolted connection according to EN 1993-1-8 is the following:

Determination of the resistance of the main components of the connection, i.e.:

- Column web in shear,
- Column web in compression,
- Beam flange and web in compression.

Determination of forces on the bolts based on:

- the resistance in bending of the column flange,
- the resistance in bending of the end plate,
- the tension resistance of the bolts.

The bending moment resistance is finally obtained multiplying the forces of the bolts with the corresponding distances from the level of application of the compressive force, which is considered to be the center of the beam's lower flange.

9.49.1 Resistance of the main components

9.49.1.1 Column web in shear

$$\frac{d}{t_w} = \frac{225}{11.5} = 20 < 69\varepsilon = 69 \qquad \text{6.2.6.1(1)}$$

$$A_{vc} = A - 2bt_f + (t_w + 2r)t_f = 161 - 2 \cdot 30 \cdot 2.05 +$$
$$+ (1.15 + 2 \cdot 2.7)2.05 = 51.4\ \text{cm}^2 > \eta h_w t_w = 1 \cdot 32 \cdot 1.15 = 36.8\ \text{cm}^2$$

$$V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\gamma_{M0}} = \frac{0.9 \cdot 23.5 \cdot 51.4}{\sqrt{3} \cdot 1.0} = 628kN$$ Eq. 6.7

9.49.1.2 Column web in compression 6.2.6.2

$$b_{\text{eff},c,wc} = t_{fb} + 2\sqrt{2}\alpha_p + s_p + 5(t_{fc} + r_c) =$$
$$= 11.5 + 2\sqrt{2} \cdot 6 + 2 \cdot 20 + 5(20.5 + 27) = 306 \text{ mm}$$

in which $s_p = 2 \cdot t_p$

$$\beta = 1$$ Tab. 5.4

$$\omega = \omega_1 = \frac{1}{\sqrt{1 + 1.3(b_{\text{eff},c,wc} t_{wc}/A_{vc})^2}} =$$
$$= \frac{1}{\sqrt{1 + 1.3\left(\frac{30.6 \cdot 1.15}{51.4}\right)^2}} = 0.79$$ Tab. 6.3

$$k_{wc} = 1.0$$ 6.2.6.2(2)

$$F_{c,wc,Rd} = \frac{\omega \cdot k_{wc} \cdot b_{\text{eff}.c.wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}} =$$
$$= \frac{0.79 \cdot 1 \cdot 30.6 \cdot 1.15 \cdot 23.5}{1.0} = 653 \text{ kN}$$ Eq. 6.9+Remark

$$d_{wc} = h_c - 2(t_{fc} + r_c) = 320 - 2(20.5 + 27) = 225 \text{ mm}$$

$$\bar{\lambda}_p = 0.932\sqrt{\frac{b_{\text{eff},wc} \cdot d_{w,c} \cdot f_{y,wc}}{E \cdot t_{wc}^2}} =$$
$$= 0.932\sqrt{\frac{30.6 \cdot 22.5 \cdot 23.5}{2.1 \cdot 10^4 \cdot 1.15^2}} = 0.73 > 0.72$$ Eq. 6.13c

$$\rho = \frac{\bar{\lambda}_p - 0.2}{\bar{\lambda}_p^2} = \frac{0.73 - 0.2}{0.73^2} = 0.99 < 1, \quad \text{and}$$ Eq. 6.13b

$$F_{c,wc,Rd} = \frac{\omega \cdot k_{wc} \cdot \rho \cdot b_{\text{eff},c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M1}} =$$
$$= \frac{0.79 \cdot 1 \cdot 0.99 \cdot 33.6 \cdot 1.15 \cdot 23.5}{1.0} = 689 \text{ kN}$$ Eq. 6.9

9.49.1.3 Beam flange and web of the beam in compression 6.2.6.7

$$M_{c,Rd} = \frac{M_{pl,b}}{\gamma_{M0}} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{804 \cdot 23.5}{1.0} = 18894 \text{ kNcm} = 189 \text{ kNm}$$

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h_b - t_{fb}} = \frac{18894}{33 - 1.15} = 593 \text{ kN}$$ Eq. 6.21

9.49.2 Column flange in bending

6.2.6.4

9.49.2.1 Upper 1st bolt-row

$$e_1 = 30 \text{ mm}, \quad e = 40 \text{ mm}, \quad p = 95 \text{ mm}$$

$$m = \frac{w - t_{wc}}{2} - 0.8 r_c = \frac{120 - 11.5}{2} - 0.8 \cdot 27 = 32.7 \text{ mm}$$

Fig. 6.8

Effective length for individual bolt-row Tab. 6.4

Circular patterns of failure

$$l_{\text{eff},cp} = 2\pi m = 2 \cdot \pi \cdot 32.7 = 205 \text{ mm}$$
$$= \pi m + 2e_1 = \pi \cdot 32.7 + 2 \cdot 30 = 163 \text{ mm}$$

Non-circular patterns of failure

$$l_{\text{eff},nc} = 4m + 1.25e = 4 \cdot 32.7 + 1.25 \cdot 40 = 181 \text{ mm}$$
$$= 2m + 0.625e + e_1 = 2 \cdot 32.7 + 0.625 \cdot 40 + 30 = 120 \text{ mm}$$

Thus: $l_{\text{eff},cp} = 163$ mm, $l_{\text{eff},nc} = 120$ mm,

$$l_{\text{eff},1} = 120 \text{ mm} < 163\text{mm}$$
$$l_{\text{eff},2} = 120 \text{ mm}$$

Effective length for bolt-group

$$l_{\text{eff},cp} = \pi m + p = \pi \cdot 32.7 + 95 = 198 \text{ mm}$$
$$= p + 2e_1 = 95 + 2 \cdot 30 = 155 \text{ mm}$$

Tab. 6.4

$$l_{\text{eff},nc} = 2m + 0.625e + 0.5p =$$
$$= 2 \cdot 32.7 + 0.625 \cdot 40 + 0.5 \cdot 95 = 138 \text{ mm}$$
$$= e + 0.5p = 40 + 0.5 \cdot 95 = 87 \text{ mm}$$

Thus: $l_{\text{eff},cp} = 155$ mm, $l_{\text{eff},nc} = 87$ mm,

$$\sum l_{\text{eff},1} = 87 \text{ mm} < 155 \text{ mm}$$
$$\sum l_{\text{eff},2} = 87 \text{ mm}$$

Tab. 6.4

Individual bolt-row

$$M_{pl,1.Rd} = M_{pl,2,Rd} = 0,25 \sum l_{\text{eff}} t_f^2 f_y / \gamma_{M0} =$$
$$= 0.25 \cdot 12 \cdot 2.05^2 \cdot 23.5/1.0 = 296 \text{ kNcm}$$

Tab. 6.2

$$n = e_{\min} = 40 \text{ mm} < 1.25\, m = 1.25 \cdot 32.7 = 40.9 \text{ mm}$$

Tab. 6.2

$$F_{t,Rd} = F_{t,Rd} = \frac{k_2 A_s f_{ub}}{\gamma_{M2}} = \frac{0.9 \cdot 2.45 \cdot 80}{1.25} = 141 \text{ kN}$$

Tab. 3.4

$$\sum F_{t,Rd} = 2F_{t,Rd} = 2 \cdot 141 = 282 \text{ kN}$$

For M 20 bolts, height of head 13 mm, height of nut 16 mm, washer thickness 8 mm, bolt length (= thickness of connected plates) $20 + 20.5 = 40.5$ mm:

$$L_b = 40.5 + 8 + \frac{1}{2}(13 + 16) = 63 \text{ mm}$$ Tab. 6.2

$$L_b^* = \frac{8.8 m^3 A_s}{l_{\text{eff}} t_f^3 c} = \frac{8.8 \cdot 3.27^3 \cdot 2.45}{12 \cdot 2.05^3} = 7.29 \text{ cm} > 6.3 \text{ cm}$$

Therefore, prying forces are developed

$$F_{T,1.Rd} = \frac{4M_{pl,1,Rd}}{m} = \frac{4 \cdot 296}{3.27} = 362 \text{ kN}$$ Tab. 6.2

$$F_{T,2.Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n} = \frac{2 \cdot 296 + 4 \cdot 282}{3.27 + 4} = 236 \text{ kN}$$

$$F_{T,3.Rd} = \sum F_{t,Rd} = 282 \text{ kN}$$

The final resistance is the least of the above values: $F_{T,Rd} = 236$ kN.

9.49.2.2 Inner, 2nd bolt-row

$$e = 40 \text{ mm}, p = 95 \text{ mm},$$ Fig. 6.8

$$m = 32.7 \text{ mm}$$ Tab. 6.4

Effective length for individual bolts

$$l_{\text{eff},cp} = 2\pi \text{ m} = 2 \cdot \pi \cdot 32.7 = 205 \text{ mm}$$

$$l_{\text{eff},nc} = 4 \text{ m} + 1.25e = 4 \cdot 32.7 + 1.25 \cdot 40 = 181 \text{ mm}$$

$$l_{\text{eff},1} = 181 \text{ mm} < 205 \text{ mm}$$

$$l_{\text{eff},2} = 181 \text{ mm}$$

Effective length for group of bolts Tab. 6.4

$$l_{\text{eff},cp} = 2p = 2 \cdot 95 = 190 \text{ mm}$$

$$l_{\text{eff},nc} = p = 95 \text{ mm}$$

$$\sum l_{\text{eff},1} = 95 \text{ mm} < 190 \text{ mm}$$

$$\sum l_{\text{eff},2} = 95 \text{ mm}$$

For individual bolt-row

$$M_{pl,1.Rd} = M_{pl,2.Rd} = 0.25 \cdot 18.1 \cdot 2.05^2 \cdot 23.5/1.0 = 447 \text{ kNcm}$$ Tab. 6.2

$$F_{T,1,Rd} = \frac{4 \cdot 447}{3.27} = 547 \text{ kN}$$ Tab. 6.2

$$F_{T,Rd} = \frac{2 \cdot 447 + 4 \cdot 282}{3.27 + 4} = 278 \text{ kN}$$

$$F_{T,3,Rd} = 282 \text{kN}$$

Thus, the final resistance is:

$$F_{T,Rd} = 278 \text{ kN}$$

9.49.2.3 1st and 2nd bolt-row (group of bolts)

$$l_{\text{eff},cp} = 155 + 190 = 345 \text{ mm}$$ 6.2.4.4(3)

$$l_{\text{eff},nc} = 87 + 95 = 182 \text{ mm}$$

$$\sum l_{\text{eff},1} = 182 \text{ mm} < 345 \text{ mm}$$ Tab. 6.4

$$\sum l_{\text{eff},2} = 182 \text{ mm}$$ Tab. 6.2

$$M_{pl,1.Rd} = M_{pl,2.Rd} = 0.25 \cdot 18.2 \cdot 2.05^2 \cdot 23.5/1.0 = 449 \text{ kNcm}$$

$$\sum F_{t,Rd} = 2 \cdot 282 = 564 kN$$ Tab. 6.2

$$F_{T,1,Rd} = \frac{4 \cdot 449}{3.27} = 549 \text{ kN}$$

$$F_{T,2,Rd} = \frac{2 \cdot 449 + 4 \cdot 564}{3.27 + 4} = 434 \text{ kN}$$

$$F_{T,3,Rd} = 564 \text{ kN}$$

Finally, $F_{T,Rd} = 434$ kN.

9.49.3 End plate in bending

6.2.6.5

9.49.3.1 Upper, 1st bolt-row, out of the beam's upper flange of the beam under tension

$$e_x = 30 \text{ mm}, \quad m_x = 40 - 0.8 \cdot 6 \cdot \sqrt{2} = 33.2 \text{ mm}$$ Fig. 6.10. 6.8

$$e_{\min} = e_x = 30 \text{ mm}$$ 6.2.6.5(3)

$$e = e_x = 30 \text{ mm}$$ Fig. 6.10 + Remark

Effective length for individual bolts

$$\begin{aligned} l_{\text{eff},cp} &= 2\pi m_x = 2 \cdot \pi \cdot 33.2 = 209 \text{ mm} \\ &= \pi m_x + w = \pi \cdot 33.2 + 120 = 224 \text{ mm} \\ &= \pi m_x + 2e = \pi \cdot 33.2 + 230 = 164 \text{ mm} \\ l_{\text{eff},nc} &= 4m_x + 1.25e_x = 4 \cdot 33.2 + 1.25 \cdot 30 = 170 \text{ mm} \\ &= e + 2m_x + 0.625e_x = 30 + 2 \cdot 33.2 + 0.625 \cdot 30 = 115 \text{ mm} \\ &= 0.5b_p = 0.5 \cdot 200 = 100 \text{ mm} \\ &= 0.5w + 2m_x + 0.625e_x = \\ &= 0.5 \cdot 120 + 2 \cdot 33.2 + 0.625 \cdot 30 = 145 \text{ mm} \end{aligned}$$ Tab. 6.6

Thus: $l_{\text{eff},cp} = 164$ mm, $l_{\text{eff},nc} = 100$ mm

$$l_{\text{eff},1} = 100 \text{ mm} < 164 \text{ mm}$$

$$l_{\text{eff},2} = 100 \text{ mm}$$

For an individual bolt-row

$$M_{pl,1.Rd} = M_{pl,2.Rd} = 0.25 \cdot 10 \cdot 2.0^2 \cdot 23.5/1.0 = 235 \text{ kNcm}$$ Tab. 6.2

$$n = e_{\min} = 30 < 1.25 \text{ m} = 1.25 \cdot 33.2 = 41.5 \text{ mm}$$ Tab. 6.2

$$F_{T,1,Rd} = \frac{4 \cdot 235}{3.32} = 283 \text{ kN}$$ Tab. 6.2

$$F_{T,2,Rd} = \frac{2 \cdot 235 + 3 \cdot 282}{3.32 + 3} = 208 \text{ kN}$$

$$F_{T,3,Rd} = 282 \text{ kN}$$

Finally $F_{T,Rd} = 208$ kN.

9.49.3.2 2^nd^ bolt-row (1^st^ row under the upper flange of the beam under tension)

$$e = 40 \text{ mm}, \quad \text{p} = 220 \text{ mm}$$

$$m = \frac{w - t_{wb}}{2} - 0.8\alpha_c\sqrt{2} = \frac{120 - 7.5}{2} - 0.8 \cdot 3 \cdot \sqrt{2} = 52.9 \text{ mm}$$ Fig. 6.10

$$m_2 = 95 - 40 - 11.5 - 0.8 \cdot 6 \cdot \sqrt{2} = 36.7 \text{ mm}$$ Fig. 6.11

$$\lambda_1 = \frac{m}{m+e} = \frac{52.9}{52.9 + 40} = 0.57$$ Fig. 6.11

$$\lambda_2 = \frac{m_2}{m_2 + e} = \frac{36.7}{52.9 + 40} = 0.40$$ Fig. 6.11

$$\alpha = 5.5$$ Fig. 6.11

Effective length for individual bolts Tab. 6.6

$$l_{\text{eff},cp} = 2\pi m = 2 \cdot \pi \cdot 52.9 = 332 \text{ mm}$$

$$l_{\text{eff},nc} = \alpha m = 5.5 \cdot 52.9 = 291 \text{ mm}$$

$$l_{\text{eff},1} = 291 \text{ mm} < 332 \text{ mm}$$

$$l_{\text{eff},2} = 291 \text{ mm}$$

Effective length for group of bolts Tab. 6.6

$$l_{\text{eff},cp} = \pi m + p = \pi \cdot 52.9 + 220 = 386 \text{ mm}$$

$$l_{\text{eff},nc} = 0.5p + \alpha m - (2m + 0.625e) =$$
$$= 0.5 \cdot 220 + 5.5 \cdot 52.9 - (2 \cdot 52.9 + 0.625 \cdot 40) = 270 \text{ mm}$$

$$\sum l_{\text{eff},1} = 270 \text{ mm} < 386 \text{ mm}$$

$$\sum l_{\text{eff},2} = 270 \text{ mm}$$

Remark 27. The above value of l_{eff} is normally used to determine the tensile force of the 2^nd^ and 3^rd^ bolt-rows, considered as a group of bolt-rows. However, since the force of the third bolt-row is calculated by another procedure (see paragraph 7), the value of l_{eff} is not further used.

For an individual bolt-row

$$M_{pl,1.Rd} = M_{pl,2.Rd} = 0.25 \cdot 29.1 \cdot 2.0^2 \cdot 23.5/1.0 = 684\ \text{kNcm} \qquad \text{Tab. 6.2}$$

$$F_{T,1,Rd} = \frac{4 \cdot 684}{5.29} = 517\ \text{kN}$$

$$F_{T,2,Rd} = \frac{2 \cdot 684 + 4 \cdot 282}{5.29 + 4} = 269\ \text{kN}$$

$$F_{T,3,Rd} = 282\ \text{kN}$$

and finally $F_{T,Rd} = 269$ kN.

9.49.4 Beam web under tension, 2^{nd} bolt-row — 6.2.6.8

$$F_{t2.wb,Rd} = b_{\text{eff},t,wb} t_{wb} f_{y,wb} / \gamma_{M0} = 29.1 \cdot 0.75 \cdot 23.5/1.0 = 513\ \text{kN} \qquad \text{Eq. 6.22}$$

9.49.5 Column web under tension, 2^{nd} bolt-row — 6.2.6.3

$$\omega = \frac{1}{\sqrt{1 + 1.3(b_{\text{eff}} t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1 + 1.3\left(\frac{18.1 \cdot 1.15}{51.4}\right)^2}} = 0.91 \qquad \text{Tab. 6.3}$$

$$F_{t2.wc,Rd} = \omega \cdot b_{\text{eff},t,wc} t_{wc} f_{y,wc} / \gamma_{M0} = 0.91 \cdot 18. \cdot 1.15 \cdot 23.5/1.0 =$$

$$= 445\ \text{kN} \qquad \text{Eq. 6.15}$$

9.49.6 Column web under tension, 1^{st} and 2^{nd} bolt-row — 6.2.6.3

$$b_{\text{eff}} = 87 + 95 = 182\ \text{mm}$$

$$\omega = \frac{1}{\sqrt{1 + 1.3\ \left(\frac{18.2 \cdot 1.15}{51.4}\right)^2}} = 0.91$$

$$F_{t,wc,Rd} = \omega \cdot b_{\text{eff},t,wc} b_{\text{eff}} t_{wc} f_{y,wc} / \gamma_{M0} =$$

$$= 0.91 \cdot 18.2 \cdot 1.15 \cdot 23.5/1.0 = 448\ \text{kN} \qquad \text{Eq. 6.15}$$

9.49.7 Bolts' forces — 6.2.7.2

1^{st} bolt-row

$F_{t1.Rd} = \min\{F_{t,cf,Rd}, F_{t,ep,Rd}\} = \min\{236\ \text{kN}, 208\ \text{kN}\} = 208$ kN (see paragr. 9.49.2.1, 9.49.3.1)

2nd bolt-row

$$F_{t2.Rd} = V_{wp,Rd}/\beta - F_{t1.Rd} = \frac{628}{1} - 208 = 420\,\text{kN} \quad \text{6.2.7.2(7)}$$

$$= F_{c,wc,Rd} - F_{t1.Rd} = 689 - 208 = 481\,\text{kN} \quad \text{(see paragr. 9.49.1.2)}$$

$$= F_{c,fb,Rd} - F_{t1.Rd} = 593 - 208 = 385\,\text{kN} \quad \text{(see paragr. 9.49.1.3)}$$

$$= F_{t2.fc,Rd} = 278\,\text{kN} \quad \text{(see paragr. 9.49.2.2)}$$

$$= F_{t2.wc,Rd} = 445\,\text{kN} \quad \text{(see paragr. 9.49.5)}$$

$$= F_{t2.ep,Rd} = 269\,\text{kN} \quad \text{(see paragr. 9.49.3.2)}$$

$$= F_{t(1+2),fc,Rd} - F_{t1.Rd} = 434 - 208 = 226\,\text{kN} \quad \text{(see paragr. 9.49.2.3)}$$

$$= F_{t(1+2),wc,Rd} - F_{t1.Rd} = 448 - 208 = 240\,\text{kN} \quad \text{(see paragr. 9.49.6)}$$

Therefore: $\min F_{t2.Rd} = 226$ kN.

3 rd bolt-row

Due to the large distance between 2nd and 3rd rows, the force of the 3rd row is obtained from the resistance of the column web in shear. Therefore:

$$F_{t3.Rd} = V_{wp,Rd}/\beta - F_{t1.Rd} - F_{t2.Rd} = \frac{628}{1} - 208 - 226 = 194\,\text{kN} \quad \text{6.2.7.2(7)}$$

Remark 28. The force of the 3rd bolt-row could be determined using the same procedure as 1st and 2nd rows. However, since this force is limited by the resistance of the column web in shear, which gives in this example a smaller force for this bolt-row, the previous procedure was followed in this example.

9.49.8 Bending moment resistance of the connection

Distances from the compression point (lower beam flange) Fig. 6.15

$$h_1 = 420 - 30 - 20 - \frac{11,5}{2} - = 364\,\text{mm}$$

$$h_2 = 364 - 95 = 269\,\text{mm}$$

$$h_3 = 269 - 220 = 49\,\text{mm}$$

In Fig. 9.74 the distribution of tension forces that act on the bolts is presented. It can be observed that the distribution of forces on the bolts is almost uniform (non-linear).

Bending moment resistance of the connection

$$M_{j,Rd} = \sum_{r=1}^{3} h_r F_{tr,Rd} = 0.364 \cdot 208 + 0.269 \cdot 226 + 0.049 \cdot 194 = 146\,\text{kNm} \quad \text{Eq. 6.25}$$

It is noticed that:

$$M_{j,Rd} = 146\,\text{kNm} < M_{c,Rd} = 189\,\text{kNm.(see paragr. 9.49.1.3)}.$$

Since the bending moment resistance of the connection is less than the bending moment resistance of the beam, the connection is classified as a connection of partial strength.

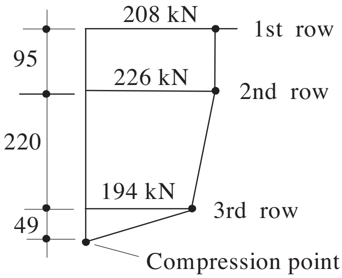


Fig. 9.74. Distribution of the bolts' tensile forces

9.49.9 Design shear of the connection

3.4.1

The bolted connection belongs to Category A (bearing type)

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A_s}{\gamma_{M2}} = \frac{0.6 \cdot 80 \cdot 2.45}{1.25} = 94 \text{ kN}$$

Bearing resistance of bolts

$$e_2 = 40 \text{ mm} > 1.2 d_0 = 1.2 \cdot 22 = 26 \text{ mm}$$ Tab. 3.3

$$p = 95 \text{ mm} > 2.2 d_0 = 2.2 \cdot 22 = 48 \text{ mm}$$

$$k_1 = \min\left\{2.8\frac{e_2}{d_0} - 1.7, 2.5\right\} = \min\left\{2.8\frac{40}{22} - 1.7, 2.5\right\} = 2.5$$

$$\alpha_b = \min\left\{\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1.0\right\} =$$

$$= \min\left\{\frac{30}{3 \cdot 22}, \frac{95}{3 \cdot 22} - \frac{1}{4}, \frac{80}{36}, 1.0\right\} = 0.45$$ Tab. 3.4

$$t = \min\{20.20.5\} = 20 \text{ mm}$$

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d\, t}{\gamma_{M2}} = \frac{2.5 \cdot 0.45 \cdot 36 \cdot 2.0 \cdot 2.0}{1.25} = 130 \text{ kN}$$

Thus: $\min(F_{v,Rd}, F_{b,Rd}) = F_{v,Rd} = 94$ kN and for one bolt-row (two bolts):

$$F_{v,Rd} = 2 \cdot 94 = 188 \text{ kN}$$

The reduced shear resistance of the bolts due to the simultaneous presence of tension is given by the relation:

$$F_{v,sd} = F_{v,Rd}\left(1 - \frac{F_{t,sd}}{1.4 F_{t,Rd}}\right)$$ Tab. 3.4

1st bolt-row

$$F_{v,sd} = 188 \cdot \left(1 - \frac{208}{1.4 \cdot 282}\right) = 89 \text{ kN}$$

2nd bolt-row

$$F_{v,sd} = 188 \cdot \left(1 - \frac{226}{1.4 \cdot 282}\right) = 80\,\text{kN}$$

3rd bolt-row

$$F_{v,sd} = 188 \cdot \left(1 - \frac{194}{1.4 \cdot 282}\right) = 96\,\text{kN}$$

Therefore, the design shear force that the connection can resist simultaneously with the bending moment determined in paragr. 9.49.8 is:

$$V_{Rd} = 89 + 80 + 96 = 265\text{ kN}$$

Design shear of the beam

$$A_{vb} = 62.6 - 2 \cdot 16 \cdot 1.15 + (0.75 + 2 \cdot 1.8)1.15 = 30.8\text{ cm}^2$$

$$V_{wb,Rd} = \frac{f_{y,wb} A_{vb}}{\sqrt{3}\gamma_{M0}} = \frac{23.5 \cdot 30.8}{\sqrt{3} \cdot 1.0} = 418\text{ kN} > 265\text{ kN}$$

It is noticed that the design shear of the connection is less than the design shear of the beam when the full bending moment of the connection is developed.

9.49.10 Rotational stiffness of the joint for $M_{j,sd} = 100$ kNm

Stiffness coefficients k_i to be considered:

$$k_1,\ k_2,\ k_{eq}$$ Tab. 6.10

where k_{eq} is based on coefficients k_3, k_4, k_5, k_{10} 6.3.3.1(4)

9.49.10.1 Coefficient k_1 (column web in shear)

The lever arm is taken approximately as: Fig. 6.15

$$z = 33 - 1.15 = 31.85\text{ cm}$$

$$\beta = 1$$ Tab. 5.4

$$k_1 = \frac{0.38 A_{vc}}{\beta z} = \frac{0.38 \cdot 51.4}{1.0 \cdot 31.85} = 0.613\text{ cm}$$ Tab. 6.11

9.49.10.2 Coefficient k_2 (column web in compression)

$$d_c = 32 - 2 \cdot 2.05 = 27.9\text{ cm}$$

$$k_2 = \frac{0.7 b_{\text{eff},c,wc} t_{wc}}{d_c} = \frac{0.7 \cdot 30.6 \cdot 1.15}{27.9} = 0.883\text{ cm}$$ Tab. 6.11

Coefficients k_3, k_4, k_5 and k_{10} are calculated separately for each bolt-row.

9.49.10.3 Coefficient k_3 *(column web in tension)*

$$k_3 = \frac{0.7 b_{\text{eff},t,wc}}{d_c} \quad \text{Tab. 6.11}$$

$$b_{\text{eff},t,wc} = \min\{120, 87\} = 87 \text{ mm} \quad (\text{seeparagr.2.1})$$

1st row $k_{3.1} = \frac{0.70 \cdot 8.7 \cdot 1.15}{27.9} = 0.251$ cm

$$b_{\text{eff},t,wc} = \min\{181, 95\} = 95 \text{ mm}$$

2nd row $k_{3.2} = \frac{0.70 \cdot 9.5 \cdot 1.15}{27.9} = 0.274$ cm (see paragr. 2.2)

9.49.10.4 Coefficient k_4 *(column flange in bending)*

$$k_4 = \frac{0.90 l_{\text{eff}} t_{fc}^3}{m^3} \quad \text{Tab. 6.11}$$

1st row $k_{4.1} = \frac{0.9 \cdot 8.7 \cdot 2.05^3}{3.27^3} = 1.929$ cm
2nd row $k_{4.2} = \frac{0.9 \cdot 9.5 \cdot 2.05^3}{3.27^3} = 2.107$ cm

9.49.10.5 Coefficient k_5 *(end plate in bending)*

$$k_5 = \frac{0.9 l_{\text{eff}}}{m^3} t_p^3 \quad \text{Tab. 6.11}$$

$$\text{1st row } k_{5.1} = \frac{0.9 \cdot 10 \cdot 2^3}{3.32^3} = 1.968 \text{ cm}$$

$$\lambda_{\text{eff}} = \min\{291, 270\} = 270 \text{ mm} \quad (\text{paragraph 9.49.3.2})$$

$$\text{2nd row } k_{5.2} = \frac{0.9 \cdot 27 \cdot 2^3}{5.29^3} = 1.313 \text{ cm}$$

9.49.10.6 Coefficient k_{10} *(bolts in tension)*

$$L_b = 20.5 + 20 + 8 + \frac{1}{2} \cdot (13 + 16) = 63 \quad (\text{paragr. 9.49.2.2})$$

$$k_{10.1} = k_{10.2} = 1.6 A_s / L_b = 1.6 \cdot 2.45 / 6.3 = 0.622 \text{ cm} \quad \text{Tab. 6.11}$$

9.49.10.7 Total stiffness

Eq. 6.30

Equivalent stiffness of individual rows

$$k_{\text{eff},1} = \frac{1}{\sum_{i=3,4,5,10} \frac{1}{k_{i,1}}} = \frac{1}{\frac{1}{0.251} + \frac{1}{1.929} + \frac{1}{1.968} + \frac{1}{0.622}} = 0.151 \text{ cm}$$

$$k_{\text{eff},2} = \frac{1}{\sum_{i=3,4,5,7} \frac{1}{k_{i,2}}} = \frac{1}{\frac{1}{0.274} + \frac{1}{2.107} + \frac{1}{1.313} + \frac{1}{0.622}} = 0.154 \text{ cm}$$

On the safety side, it is approximately taken:

$$k_{\text{eff},3} = k_{\text{eff},2} = 0.154 \text{ cm}$$

Lever arm

Eq. 6.31

$$z_{eq} = \frac{\sum k_{\text{eff},r} h_r^2}{\sum k_{\text{eff},r} h_r} = \frac{0.151 \cdot 36.4^2 + 0.154 \cdot 26.9^2 + 0.154 \cdot 4.9^2}{0.151 \cdot 36.4 + 0.154 \cdot 26.9 + 0.154 \cdot 4.9} = 30.33 \text{ cm}$$

This value does not differ significantly from the value of z, that has been used for the determination of coefficient k_1.

Eq. 6.29

$$k_{eq} = \frac{\sum k_{\text{eff},r} h_r}{z_{eq}} = \frac{0.151 \cdot 36.4 + 0.154 \cdot 26.9 + 0.154 \cdot 4.9}{30.35} = 0.342 \text{ cm}$$

$$\sum \frac{1}{k_i} = \frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_{eq}} = \frac{1}{0.613} + \frac{1}{0.883} + \frac{1}{0.342} = 5.69 \text{ cm}^{-1}$$

$\psi = 2.7$

Tab. 6.8

$$\frac{2}{3} M_{j,Rd} = M_{j,Rd} = \frac{2}{3} \cdot 146 = 97 \text{ kNm}$$

and

$$\frac{2}{3} M_{j,Rd} < M_{j,Ed} = 100 \text{ kNm} < M_{j,Rd}$$

Eq. 6.28b

$$\mu = \left[\frac{1.5 M_{j,Ed}}{M_{j,Rd}}\right]^{\psi} = \left[\frac{1.5 \cdot 100}{146}\right]^{2.7} = 1.08 > 1$$

Eq. 6.27

$$S_j = \frac{E z^2}{\mu \sum \frac{1}{k_i}} = \frac{2.1 \cdot 10^4 \ \cdot 30.33^2}{1.08 \cdot 5.69} = 3.15 \cdot 10^6 \text{ kNcm} = 31500\text{kNm}$$

9.50 Example: Beam to column welded connection

Check the welded connection between an IPE 240 beam to a HEA 280 column as shown in Fig. 9.75. In addition, determine the rotational stiffness of the joint for the design bending moment. Steel grade S 235.

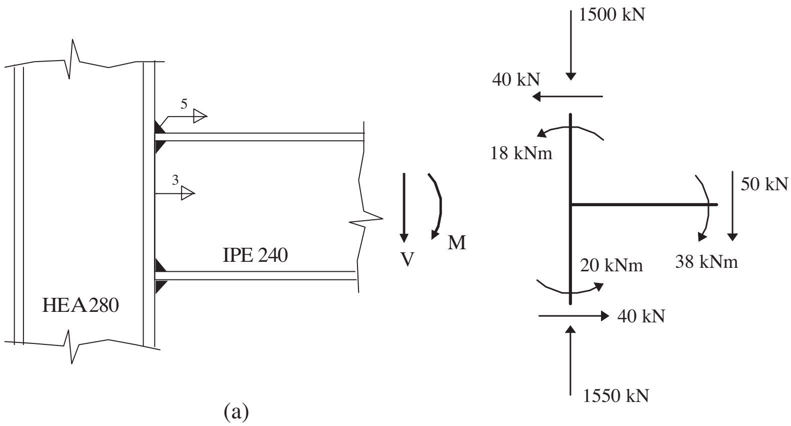


Fig. 9.75. Beam to column welded connection and design values of internal forces and moments

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.50.1 Column web in shear

6.2.6.1

$$A_{vc} = A - 2bt_f + (t_w + 2r)t_f = 97.3 - 2 \cdot 28 \cdot 1.3 + \\ + (0.8 + 22.4) \cdot 1.3 = 31.78 \text{ cm}^2 > \eta h_w t_w = 1.0 \cdot 25.4 \cdot 0.8 = 20.32 \text{ cm}^2$$

$$\frac{d}{t_w} = \frac{196}{8} = 24.5 < 63\varepsilon = 63$$ 6.2.6.1(1)

$$V_{wp,Rd} = \frac{0.9 f_{y,wc} A_{vc}}{\sqrt{3}\gamma_{M0}} = \frac{0.9 \cdot 23.5 \cdot 31.78}{\sqrt{3} \cdot 1.0} = 388 \text{ kN}$$ Eq. 6.7

Lever arm of internal forces Fig. 6.15

$$z = h - t_{fb} = 24 - 0.98 = 23.02 \text{ cm}$$

Design shear force on the column web

$$V_{wp,Ed} = \frac{M_{b1,Ed} - M_{b2,Ed}}{z} - \frac{V_{c1,Ed} - V_{c2,Ed}}{2} = \\ = \frac{3800 - 0}{23.02} - \frac{40 + 40}{2} = 125 \text{ kN}$$ Eq. 5.3

$$V_{wp,Ed} = 125 \text{ kN} < 388 \text{ kN} = V_{wp,Rd}$$

9.50.2 Column web in compression

6.2.6.2

$$b_{\text{eff},c,wc} = t_{fb} + 2\sqrt{2}\alpha_b + 5(t_{fc} + r_c) =$$
$$= 9.8 + 2 \cdot \sqrt{2} \cdot 5 + 5 \cdot (13 + 24) = 209 \text{ mm}$$

Eq. 6.10

$\beta = 1$

Tab. 5.4

$$\omega = \omega_1 = \frac{1}{\sqrt{1 + 1.3(b_{\text{eff},c,wc}t_{wc}/A_{vc})^2}} = \frac{1}{\sqrt{1 + 1.3 \cdot \left(\frac{20.9 \cdot 0.8}{31.78}\right)^2}} = 0.86$$

Tab. 6.3

Maximum compressive stress on the column

$$\sigma_{com,Ed} = \frac{N_{c1,Ed}}{A_c} + \frac{M_{c1,Ed}}{I_c}\left(\frac{h_c}{2} - t_{fc} - r_c\right) =$$
$$= \frac{1550}{97.3} + \frac{2000}{13670} \cdot \left(\frac{27}{2} - 1.3 - 2.4\right) = 17.36 \text{ kN/cm}^2$$

6.2.6.2(2)

$$\sigma_{com,Ed} = 17.36 > 0.7 f_{y,wc} = 0.7 \cdot 23.5 = 16.45 \text{ kN/cm}^2$$

$$k_{wc} = 1.7 - \sigma_{com,Ed}/f_{y,wc} = 1.7 - \frac{17.36}{23.5} = 0.96$$

Eq. 6.14

$$d_{wc} = h_c - 2(t_{fc} + r_c) = 270 - 2(1.3 + 2.4) = 196 \text{ mm}$$

$$\bar{\lambda}_p = 0.932\sqrt{\frac{b_{\text{eff},cw,wc} \cdot d_{wc} \cdot f_{y,wc}}{Et_{wc}^2}} =$$
$$= 0.932\sqrt{\frac{20.9 \cdot 19.6 \cdot 23.5}{2.1 \cdot 10^4 \cdot 0.8^2}} = 0.79$$

Eq. 6.13c

$$\rho = \frac{\bar{\lambda}_p - 0.2}{\bar{\lambda}_p^2} = \frac{0.79 - 0.2}{0.79^2} = 0.95$$

Eq. 6.13b

$$F_{c,wc,Rd} = \frac{\omega \cdot k_{wc}\rho \cdot b_{\text{eff},c,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M1}} =$$
$$= 0.86 \cdot \frac{0.96 \cdot 0.95 \cdot 20.9 \cdot 0.8 \cdot 23.5}{1.0} = 308 \text{ kN}$$

Eq. 6.9

9.50.3 Flange and web of the beam in compression

6.2.6.7

$$M_{c,Rd} = \frac{W_{pl} f_{y,b}}{\gamma_{M0}} = \frac{366 \cdot 23.5}{1.0} = 8601 \text{ kNcm}$$

5.4.6

Design shear resistance of the beam

$$A_{vb} = 39.1 - 2 \cdot 12 \cdot 0.98 + (0.62 + 2 \cdot 1.5) \cdot 0.98 = 19.1 \text{ cm}^2$$

$$V_{b,Rd} = \frac{A_{vb} f_{y,b}}{\sqrt{3}\gamma_M} = \frac{19.1 \cdot 23.5}{\sqrt{3} \cdot 1.0} = 259 \text{ kN}$$

Eq. 6.21

$$V_{b,Ed}/V_{b,Rd} = 50/259 = 0.19 < 0.5$$

Therefore no reduction of the moment $M_{c,Rd}$ is necessary due to coexistence of shear.

$$F_{c,fb,Rd} = \frac{M_{c,Rd}}{h - t_{fb}} = \frac{8601}{24 - 0.98} = 374 \text{ kN}$$

9.50.4 Column flange in bending

6.2.6.4.3

$$t_p = t_{fb}$$

$$k = \frac{t_f}{t_p}\frac{f_{y,f}}{f_{y,p}} = \frac{13}{9.8} \cdot \frac{23.5}{23.5} = 1.33 > 1$$ Eq. 4.6b

therefore, $k = 1$

Rolled section: $s = r$ Eq. 4.6c

$$b_{\text{eff},b,fc} = t_w + 2 \cdot s + 7 \cdot k \cdot t_f = 0.8 + 2 \cdot 2.4 + 7 \cdot 1 \cdot 1.3 = 14.7 \text{ cm}$$ Eq. 4.6a

but

$$b_{\text{eff},b,fc} = 14.7 \text{ cm} > b_{fb} = 12 \text{ cm}$$

and finally

$$b_{\text{eff},b,fc} = 12 \text{ cm}$$ Fig. 4.8

$$F_{fc,Rd} = b_{\text{eff},b,fc} t_{fb} f_{y,fb} / \gamma_{M0} = 12 \cdot 0.98 \cdot 23.5/1.0 = 276 \text{ kN}$$

The following criterion is satisfied:

$$b_{\text{eff}} \geqslant \frac{f_{y,p}}{f_{u,p}} \cdot b_p$$

or

$$14.7 > \frac{23.5}{36} \cdot 12 = 10.8 \text{ cm}$$ Eq. 4.7

Therefore

$$F_{fc,Rd} = 276 \text{ kN}$$

9.50.5 Column web in tension

6.2.6.3

Rolled section: $s = r_c$

$$b_{\text{eff},t,wc} = t_{fb} + 2\sqrt{2} a_b + 5(t_{fc} + r_c) = \\ = 9.8 + 2 \cdot \sqrt{2} \cdot 5 + 5 \cdot (13 + 24) = 209 \text{ mm}$$ Eq. 6.16

$\omega = 0.86$ (see paragr. 9.50.2)

$$F_{t2.wc,Rd} = \frac{\omega \cdot b_{\text{eff},t,wc} \cdot t_{wc} \cdot f_{y,wc}}{\gamma_{M0}} = \frac{0.86 \cdot 20.9 \cdot 0.8 \cdot 23.5}{1.0} = 338 \text{ kN}$$ Eq. 6.15

9.50.6 Maximum value of the couple of forces at the levels of beam flanges and beam verification

$$F_{Rd} = \min\{F_{c,wc,Rd}, F_{c,tf,Rd}, F_{t,fc,Rd}, F_{t,wc,Rd}\} = \\ = \min\{308, 374, 276, 338\}\ \text{kN} = 276\ \text{kN}$$ Fig. 6.15

Design moment resistance

$$M_{j,Rd} = F_{Rd}z = 276 \cdot 0.2302 = 63.5\ \text{kNm} > M_{Ed} = 38\ \text{kNm}$$

9.50.7 Check of welds

Steel S 235: $\beta_w = 0.8$ Tab. 4.1

Welds in the web

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = \frac{36/\sqrt{3}}{0.8 \cdot 1.25} = 20.8\ \text{kN/cm}^2$$ Eq. 4.4

$$F_{w,Rd} = f_{vw,d}a = 20.8 \cdot 0.3 \cdot 2 = 12.5\ \text{kN/cm}$$ Eq. 4.3

Design shear resistance

$$V_{w,Rd} = F_{w,Rd}l = 12.5 \cdot (24 - 2 \cdot 0.98 - 2 \cdot 1.5) = 238\ \text{kN} > V_{w,Ed} = 50\ \text{kN}$$

Welds in the flange

$$F_{w,Rd} = f_{vw,d}a = 20.8 \cdot 0.5 = 10.4\ \text{kN/cm}$$ Eq. 4.3

$$F_{f,Rd} = F_{w,Rd}l = 10.4 \cdot (2 \cdot 12 - 0.62 - 2 \cdot 1.5) = 212\ \text{kN}$$ Eq. 4.3

Force on flange welds:

$$F_{f,Ed} = \frac{M_{b,Ed}}{h_b - t_{fb}} = \frac{3800}{24 - 0.98} = 165\ \text{kN} < F_{f,Rd} = 212\ \text{kN}$$

Force on flange welds corresponding to bending moment resistance of the connection:

$$F_{Ed} = \frac{6350}{24 - 0.98} = 276\ \text{kN} > 212\ \text{kN}$$

9.50.8 Rotational stiffness of the joint for $M_{j,Ed} = 38$ kNm

Stiffness coefficients k_i to be considered:

$$k_1, \quad k_2, \quad k_3$$ Tab. 6.9

- Coefficient k_1 (column web in shear)

$$k_1 = \frac{0.38A_{vc}}{\beta z} = \frac{0.38 \cdot 31.78}{1.0 \cdot 23.02} = 0.52\ \text{cm}$$ Tab. 6.11

- Coefficient k_2 (column web in compression)

$$d_c = 27 - 2 \cdot 1.3 = 24.4 \text{ cm}$$

$$k_2 = \frac{0.7 b_{\text{eff},c,wc} t_{wc}}{d_c} = \frac{0.7 \cdot 20.9 \cdot 0.8}{24.4} = 0.48 \text{ cm}$$ Eq. 6.11

- Coefficient k_3 (column web in tension)

$$k_3 = \frac{0.7 b_{\text{eff},t,wc} t_{wc}}{d_c} = \frac{0.7 \cdot 20.9 \cdot 0.8}{24.4} = 0.48 \text{ cm}$$

$$\frac{2}{3} M_{j,Rd} = \frac{2}{3} \cdot 63.5 = 42.3 \text{ kNm}$$

It is: $M_{j,\mathrm{E}d} = 38 \text{ kNm} < \frac{2}{3}\mathrm{M_{j,Rd}}$

So, $\mu = 1$ Eq. 6.28a

$$S_j = \frac{E z^2}{\mu \sum \frac{1}{k_i}} = \frac{2.1 \cdot 10^4 \cdot 23.02^2}{1 \cdot \left(\frac{1}{0.52} + \frac{1}{0.48} + \frac{1}{0.48}\right)} = 1827 \cdot 10^3 \text{ kNcm}$$ Eq. 6.27

9.51 Example: Steel column base under axial load

Verify the capacity of a steel column base, shown in Fig. 9.76 with a HEB 300 cross-section, subjected to a design axial load $N_{Ed} = 3000$ kN. The column base plate is $500 \cdot 500 \cdot 30$ and it is placed on a $3000 \cdot 3000 \cdot 1400$ reinforced concrete foundation. Concrete class C30/37. Steel grade S 275.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.51.1 Dimensions of effective foundation

EN 1992-1. 6.7 + Fig. 6.29

$$\max b_2 = 3b_1 = 3 \cdot 500 = 1500 \text{ mm}$$

$$\max d_2 = 3d_1 = 3 \cdot 500 = 1500 \text{ mm}$$

The area $A_{c1} = b_2 \cdot d_2$ is all included in the lower area of the foundation and it is also valid:

$$h \geqslant b_2 - b_1 = 2b_1 = 1000 \text{ mm}$$

$$h \geqslant d_2 - d_1 = 2d_1 = 1000 \text{ mm}$$

Thus $A_{co} = b_1 d_1 = 500 \cdot 500 = 250 \cdot 10^3 \text{ mm}^2$

$$A_{c1} = b_2 d_2 = 1500 \cdot 1500 = 2250 \cdot 10^3 \text{ mm}^2$$

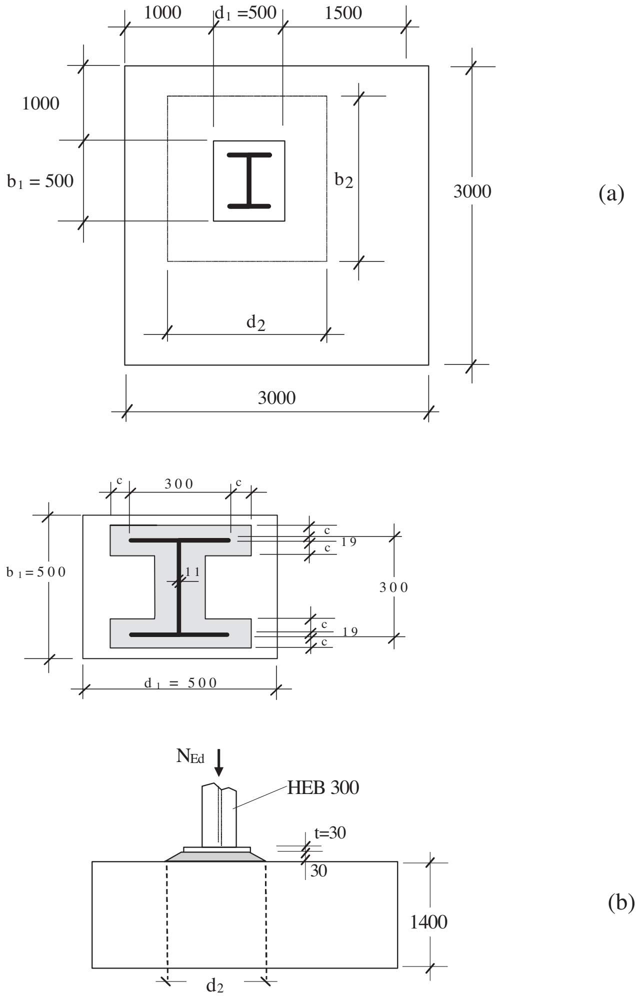


Fig. 9.76. Steel column base

9.51.2 Design resistance of concrete

The design resistance against a concentrated force is:

$$F_{Rdu} = A_{co} \cdot f_{cd} \cdot \sqrt{A_{c1}/A_{co}} \leqslant 3f_{cd} \cdot A_{co} \qquad \text{EN 1992-1. Eq. 6.63}$$

or

$$F_{Rdu} = 250 \cdot 10^3 \cdot \frac{30}{1,5} \cdot \sqrt{\frac{2250 \cdot 10^3}{250 \cdot 10^3}} = 15 \cdot 10^6 \text{ N}$$

$$\leqslant 3f_{cd}A_{co} = 3 \cdot \frac{30}{1.5} 250 \cdot 10^3 = 15 \cdot 10^6 \text{ N.}$$

Provided that the characteristic strength of the grout is not less than 0.2 times the characteristic strength of the concrete foundation and the thickness of the grout

(30 mm) is not greater than 0.2 times the smallest width of the steel base plate (i.e. 0.2 min $(d_{1.}b_1) = 100mm > 30$ mm), the foundation joint material coefficient may be taken as: $\beta_j = 2/3$. 6.2.5(7)

The design bearing strength of the joint is:

$$f_{jd} = \beta_j F_{Rdu}/(b_{\text{eff}} \cdot l_{\text{eff}}) \frac{2}{3} \cdot \frac{15 \cdot 10^6}{(250 \cdot 10^3)} == 40\,\text{N/mm}^2$$ Eq. 6.6

9.51.3 Effective base plate area

Effective bearing width:

$$c \leqslant t \left(\frac{f_y}{3 f_{jd} \gamma_{M0}} \right)^{0.5} = 30 \cdot \left(\frac{275}{3 \cdot 40 \cdot 1.0} \right)^{0.5} = 45.4\,\text{mm}$$ Eq. 6.5

Effective area (Fig. 9.76):

$$A_{\text{eff}} = (300 + 2c)^2 - (300 + 2c - t_w - -2c) \cdot (300 - 2t_f - 2c =$$
$$= (300 + 2 \cdot 45.4)^2 - (300 - 11) \cdot (300 - 2 \cdot 19 - 2 \cdot 45.4) = 103 \cdot 10^3\ \text{mm}^2$$

9.51.4 Verification of capacity in compression

$$N_{Rd} = A_{\text{eff}} f_{jd} = 103 \cdot 10^3 \cdot 40 = 4120 \cdot 10^3 N = 4120\,\text{kN} > N_{Ed} = 3000\,\text{kN}$$

Remark 29. Verification of capacity in bending for the base plate is not necessary, since the effective bearing width c (Eq. 6.5) has been calculated from the relation $M_{Ed} = M_{Rd}$.

9.52 Example: Steel column base under axial load and bending moment about the column's major principal axis

In the column base of Example 9.51. calculate the design resistance in bending about the major principal axis, which corresponds to an axial design force $N_{Ed} = 1000$ kN. The anchor bolts are 4 M 24 of 4.6 class and are placed as shown in Fig. 9.77 Steel grade S 275.

Note. In this design example, all the references in grey through the text refer to EN 1993-1-8, unless otherwise is written.

9.52.1 Effective length

Depending on the failure mode, the effective length of the base plate considering the influence of the anchor bolts, is given by the relations: Tab. 6.6

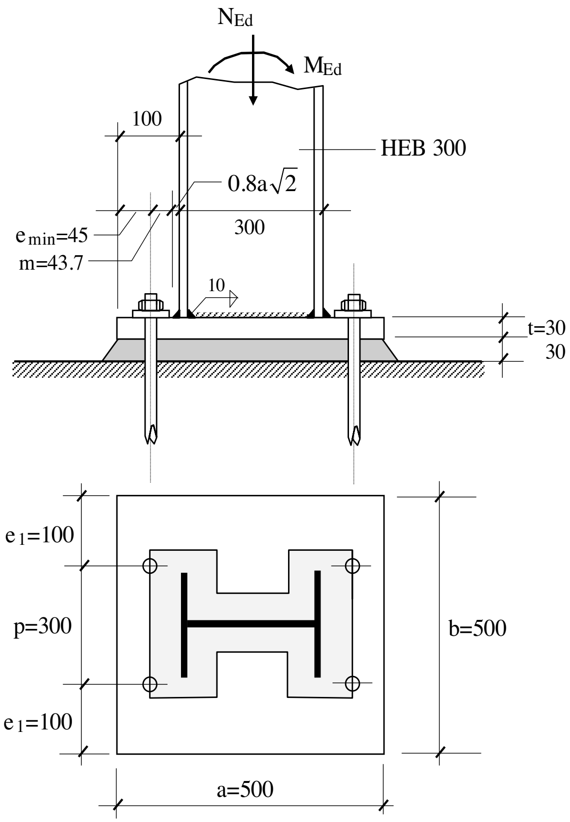


Fig. 9.77. Details of the base plate

$$\sum l_{\mathrm{eff},1} = l_{\mathrm{eff},np} = \min \begin{cases} 4m_x + 1.25e_x \\ 2m_x + 0.625e_x + e \\ 2m_x + 0.625e_x + 0.5w \\ b_p/2 \end{cases}$$

But also:

$$\sum l_{\mathrm{eff},1} \leqslant l_{\mathrm{eff},cp} = \min \begin{cases} 2\pi m_x \\ \pi m_x + 2e \\ \pi m_x + w \end{cases}$$

Fig. 6.10 + 6.8

and:

$$\sum l_{\mathrm{eff},2} = l_{\mathrm{eff},nc}$$

It is:

$$m_x = 100 - e_x - 0.8a\sqrt{2} = 100 - 45 - 0.8 \cdot 10 \cdot \sqrt{2} = 43.7 \text{ mm}$$

These relations lead to:

$$\begin{aligned} l_{\text{eff}.nc} &= 4 \cdot 43.7 + 1.25 \cdot 45 = 231.1 \text{ mm} \\ &= 100 + 2 \cdot 43.7 + 0.625 \cdot 45 = 215.5 \text{ mm} \\ &= 0.5 \cdot 300 + 2 \cdot 43.7 + 0.625 \cdot 45 = 265.5 \text{ mm} \\ &= 0.5 \cdot 500 = 250 \text{ mm} \\ l_{\text{eff}.cp} &= 2 \cdot \pi \cdot 43.7 = 274.6 \text{ mm} \\ &= \pi \cdot 43.7 + 2 \cdot 100 = 337.3 \text{ mm} \\ &= \pi \cdot 43.7 + 300 = 437.3 \text{ mm} \end{aligned}$$

Thus:

$$\sum l_{\text{eff},1} = \sum l_{\text{eff},2} = 215.5 \text{ mm}$$

9.52.2 Resistance of base plate in the side of tensioned anchor bolts

Since:

$$L_b = 8 \cdot d + t_g + t_p + t_{wa} + 0.5 t_n = 8 \cdot 24 + 30 + 30 + 8 + 0.5 \cdot 19 = 269.5 \text{ mm}$$

and

$$L_b = 269.5 \text{ mm} > \frac{8.8 \cdot m^3 \cdot A_s}{\sum l_{\text{eff},1} \cdot t_f^3} = \frac{8.8 \cdot 43.7^3 \cdot 353}{215.5 \cdot 30^3} = 44.6 \text{ mm}$$

it is:

$$\begin{aligned} M_{pl,1.Rd} &= 0.25 \sum l_{\text{eff},1} t_f^2 f_y / \gamma_{M0} = \\ &= 0.25 \cdot 215.5 \cdot 3^2 \cdot 27.5 / 1.0 = 13.3 \cdot 10^3 \text{ kNmm} \end{aligned}$$ Tab. 6.2

$$n = e_{\min} = 45mm < 1.25 \cdot m = 1.25 \cdot 43.7 = 54.6 \text{ mm}$$ Tab. 6.2

and

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_{M2}} = \frac{0.9 \cdot 40 \cdot 3.53}{1.25} = 101.65 \text{ kN}$$ Tab. 3.4

Therefore:
First and second mode of failure

$$F_{T,1-2.Rd} = \frac{2 M_{pl,1,Rd}}{m} = \frac{2 \cdot 13.3 \cdot 10^3}{43.7} = 608.7 \text{ kN}$$ Tab. 6.2

Third mode of failure

$$F_{T,3.Rd} = \sum F_{t,Rd} = 2 \cdot 101.65 = 203.3 \text{ kN}$$

and min $F_{T,Rd} = 203.3$ kN.

9.52.3 Effective area

For simultaneous coexistence of axial compressive force and bending moment, the following equilibrium equations are valid (Fig. 9.78):

$$N_{Rd} = A_{\text{eff}} f_{jd} - \sum F_{t,Rd}$$
$$M_{Rd} = \sum F_{t,Rd} r_b + A_{\text{eff}} f_{jd} r_c$$

Since $f_{jd} = 40\ \text{N}/\text{mm}^2$ (Example 9.51) the first equation leads to:

$$1000 = A_{\text{eff}} \cdot 4 - 203.3$$

or

$$A_{\text{eff}} = 300.8\ \text{cm}^2$$

From Fig. 9.78:

$$A_{\text{eff}} = x_0(2c + b_f)$$

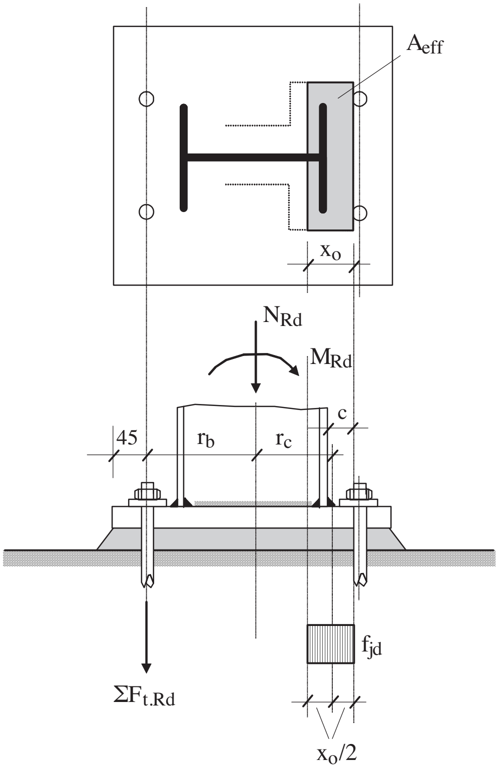


Fig. 9.78. Base plate's effective area

or

$$300.8 \cdot 10^2 = x_0 \cdot (300 + 2 \cdot 45.4)$$

and

$$x_0 = 77 \text{ mm} < t_f + 2c = 19 + 2 \cdot 45.4 = 109.8 \text{ mm}$$

9.52.4 Resistance of the joint in bending

$$r_c = \frac{300}{2} + 45.4 - \frac{77}{2} = 156.9 \text{ mm}$$

and the moment resistance that corresponds to an axial force equal to $N_{ed} = 1000$ kN is:

$$M_{Rd} = 203.3 \cdot 10^3 \cdot \left(\frac{300}{2} + 55\right) + 300.8 \cdot 10^2 \cdot 40 \cdot 156.9 =$$
$$= 230 \cdot 10^6 \text{ Nmm} = 230 \text{ kNm}$$

9.52.5 Design resistance in compression and bending of the lower column cross-section

$$M_{pl.y.Rd} = W_{pl} f_y / \gamma_{M0} = 1869 \cdot 27.5 \cdot \frac{10^{-2}}{1.0} = 514 \text{ kNm}$$
$$N_{pl,Rd} = A f_y / \gamma_{M0} = 149 \cdot 27.5 / 1.0 = 4097 \text{ kN}$$

and

$$M_{Ny.Rd} = M_{pl.y.Rd} \cdot \left(1 - \frac{N_{Ed}}{N_{pl.Rd}}\right) / (1 - 0.5 \cdot a) =$$
$$= 514 \cdot \frac{1 - \frac{1000}{4097}}{1 - 0.5 \cdot 0.23} = 439 \text{ kNm} < 514 \text{ kNm}$$

EN 1993-1-1. Eq. 6.36

where

$$a = \frac{A - 2bt_f}{A} = \frac{149 - 2 \cdot 30 \cdot 1.9}{149} = 0.23 < 0.5.$$

Since

$$V_{Ed} = 0 < 0.5 V_{pl.Rd}$$

no reduction of the moment capacity, due to shear force, is necessary.

Remark 30. This resistance is larger than the joint resistance.

In case of a V_{Ed} different than 0, the requirements of EN 1993-1-8. Tab 3.4 should additionally be satisfied for the anchor bolts.