	Project:	Doc. n.:	
	Description:	Rev.:	date

CHS member design for axial force EC3

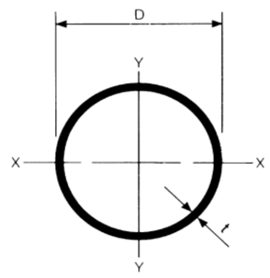
ELEMENT : CHS_design_axial_force

Material properties

Steel grade		**S 275**		Partial factor for resistance	
Yeld strength	f_y=	275	MPa	γ_{M0} =	**1,00**
Ultimate tensile strength	f_u=	430	MPa	γ_{M1} =	**1,00**
Young's modules	E=	210000	MPa	γ_{M2} =	**1,25**
Reference code	EN 1993-1-1				

Section properties

				1
type		**CHS**		10
section		**CHS 219,1x10**		
diameter	d=	219,1	mm	
thickness	t=	10	mm	
area	A=	66	cm^2	
secondo moment of area	I=	3598	cm^4	
radius of inertia	i=	7,4	cm	
	ε=	0,92		
ratio for local buckling	d/t=	21,91		
classification class		1		
buckling length about axis	L_o=	**7,8**	m	
weight	p=	51,6	kg/m	



Sollecitazioni di progetto - SLU

axial force - tension	N_{Ed+}=	**783**	kN
axial force - compression	N_{Ed-}=	**-783**	kN

Check

Tension check						
design tensile resistance	$N_{pl,Rd} = A\ f_y/\gamma_{M0}$ =	1806	kN			
check	$N_{Ed+} \leq N_{pl,Rd}$	783	**kN ≤**	***1806***	*0,43*	***OK***
buckling check						
slenderness ratio	$\lambda=L_0/i$=	106	**<**	**200** (slenderness limit)	*0,53*	***OK***
slenderness ratio	λ_1=	86,8				
non dimensional slenderness	$\overline{\lambda} = \lambda / \lambda_1$	1,22				
buckling curve a	α=	0,21				
value	Φ=	1,35				
reduction factor	χ=	0,52				
flexural buckling resistance	$N_{b,Rd} = \chi\ A\ f_y/\gamma_{M1}$ =	940	kN			
check	Fxt $\leq N_{b,Rd}$	783	**kN ≤**	***940***	*0,83*	***OK***

Partial factors for resistance

6.1(1)

γ_{M0} = 1.0

γ_{M1} = 1.0

BSEN 1993-1-8 Table 2.1

γ_{M2} = 1.25 (for shear)

Trial section

Check diagonal bracing member, 'bk' (worst case).

Steel Designers' Manual

Try: 219.1 × 10.0 mm thick Circular Hollow Section (CHS), grade S275 steel

Section Properties

Area	A	= 65.7 cm²
Second moment of area	I	= 3598 cm⁴
Radius of gyration	i	= 7.40 cm
Thickness	t	= 10.0 mm
Ratio for local Buckling	d / t	= 21.9

Material properties

Table 3.1

As $t \leq$ 40 mm, for S275 steel

Yield strength f_y = 275 N/mm²

3.2.6 (1)

Young's modules E = 210 kN/mm²

5.5

Section classification

Table 5.2

Class 1 limit for section in compression, d/t ≤ 50 ε^2

$\varepsilon = (235/fy)^{0.5}$, fy = 275 N/mm², ε = 0.92, ε^2 = 0.85

d/t ≤ 50(0.85) = 42.5 Therefore, 21.9 < 42.5

For axial compression the section is Class 1.

Design of axially loaded compression members

Cross sectional resistance to axial compression

6.2.4(1) Eq. 6.9

Basic requirement $$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

N_{Ed} is the design value of the applied axial force
$N_{Ed} = F_{bk}$ = 783 kN

$N_{c,Rd}$ is the design resistance of the cross-section for uniform compression

6.2.4(2) Eq. 6.10

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}}$$ (For Class 1, 2 and 3 cross-sections)

$$N_{c,Rd} = \frac{6570 \times 275}{1.0} \times 10^{-3} = 1807 \text{ kN}$$

Output: Compressive resistance $N_{c,Rd}$ = 1807 kN

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{783}{1807} = 0.43 < 1.0$$

Therefore, the compressive resistance of the cross section is adequate.

Flexural buckling resistance

6.3.1.1(1) Eq. 6.46

For a uniform member under axial compression the basic requirement is:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

	$N_{b,Rd}$ is the design buckling resistance and is determined from:	
6.3.1.1(3) Eq. 6.47	$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ (For Class 1, 2 and cross-sections)	
6.3.1.2(1)	χ is the reduction factor for the buckling curve and may be determined from Figure 6.4.	
Table 6.2	For hot finished CHS in grade S275 steel use buckling curve *a*	Use buckling curve *a*
	For flexural buckling the slenderness is determined from:	
6.3.1.3(1) Eq. 6.50	$\overline{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right)$ (For Class 1, 2 and 3 cross-sections)	

	Where: L_{cr} is the buckling length As the bracing member is pinned at both ends: $L_{cr} = L = \sqrt{5000^2 + 6000^2} = 7810$ mm	$L_{cr} = 7810$ mm
	$\lambda_1 = 93.9\varepsilon$	
Table 5.2	$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{275}} = 0.924$	
	$\lambda_1 = 93.9 \times 0.924 = 86.8$	$\lambda_1 = 86.8$
6.3.1.3(1) Eq. 6.50	$\overline{\lambda} = \left(\frac{7810}{74}\right) \times \left(\frac{1}{86.8}\right) = 1.22$	$\overline{\lambda} = 1.22$
Figure 6.4	For $\overline{\lambda} = 1.22$ and buckling curve *a* $\chi = 0.51$	$\chi = 0.51$
	Therefore,	Flexural buckling resistance
6.3.1.1(3) Eq. 6.47	$N_{b,Rd} = \frac{0.51 \times 65.7 \times 10^2 \times 275}{1.0} \times 10^{-3} = 921$ kN	$N_{b,Rd} = 921$ kN
6.3.1.1(1) Eq. 6.46	$\frac{N_{Ed}}{N_{b,Rd}} = \frac{783}{921} = 0.85 < 1.0$	
	Therefore, the flexural buckling resistance of the section is adequate.	
6.2.3	**Design of axially loaded tension member and connection**	
	When the wind is applied in the opposite direction, the bracing member considered above will be loaded in tension. Therefore, check section for the same magnitude of loading.	
	Cross sectional resistance of the CHS to axial tension	
6.2.3(1)	Basic requirement $\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$	
	N_{Ed} is the design value of the applied axial tension $N_{Ed} = F_{bk} = 783$ kN	
	$N_{t,Rd}$ is the design tensile resistance of the cross-section	
	For the CHS $N_{t,Rd} = N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}}$	
	$N_{t,Rd} = \frac{6570 \times 275}{1.0} \times 10^{-3} = 1807$ kN	Tensile resistance of CHS $N_{t,Rd} = 1807$ kN