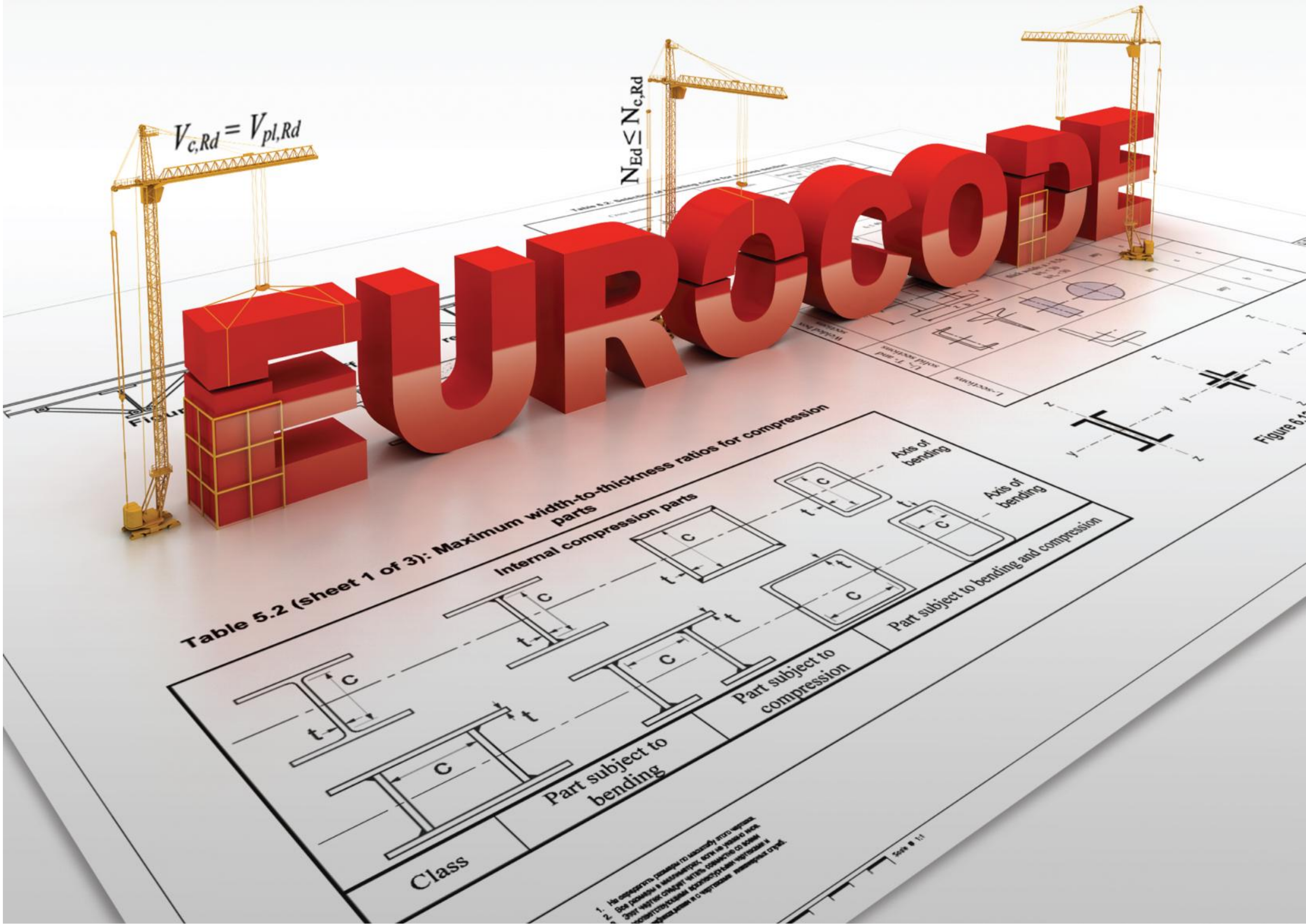
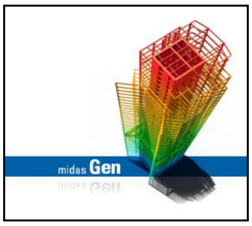
Design Examples using midas Gen to Eurocode 3

Integrated Design System for Building and General Structures



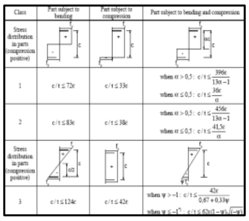
Introduction

This design example book provides a comprehensive guide for steel design as per Eurocode3-1-1:2005. Specifically, this guide will review the design algorithms implemented in midas Gen, and go through detailed verification examples and design tutorials. This book is helpful in understanding the Eurocode design concept and verifying design results using midas Gen.



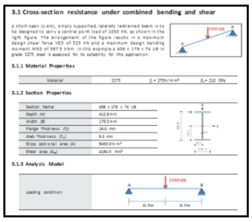
CHAPTER 1 Why midas Gen

This chapter describes the main features and advantages of midas Gen and showcases prominent project applications.



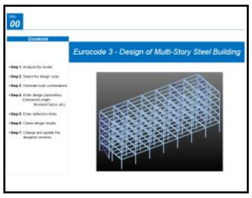
CHAPTER 2 Steel Design Algorithms

This chapter discusses the general design concept of EN1993-1-1 and how it has been implemented in midas Gen. This enables the user to understand the equations, formulas, program limitations and development scope of the midas Gen design features.



CHAPTER 3 Verification Examples

This chapter provides comparative results between design reports generated from midas Gen and design examples from reference books. Numerous worked examples for EN1993-1-1:2005 has been used to verify design results from midas Gen. 17 steel examples of beam and column members has been included.



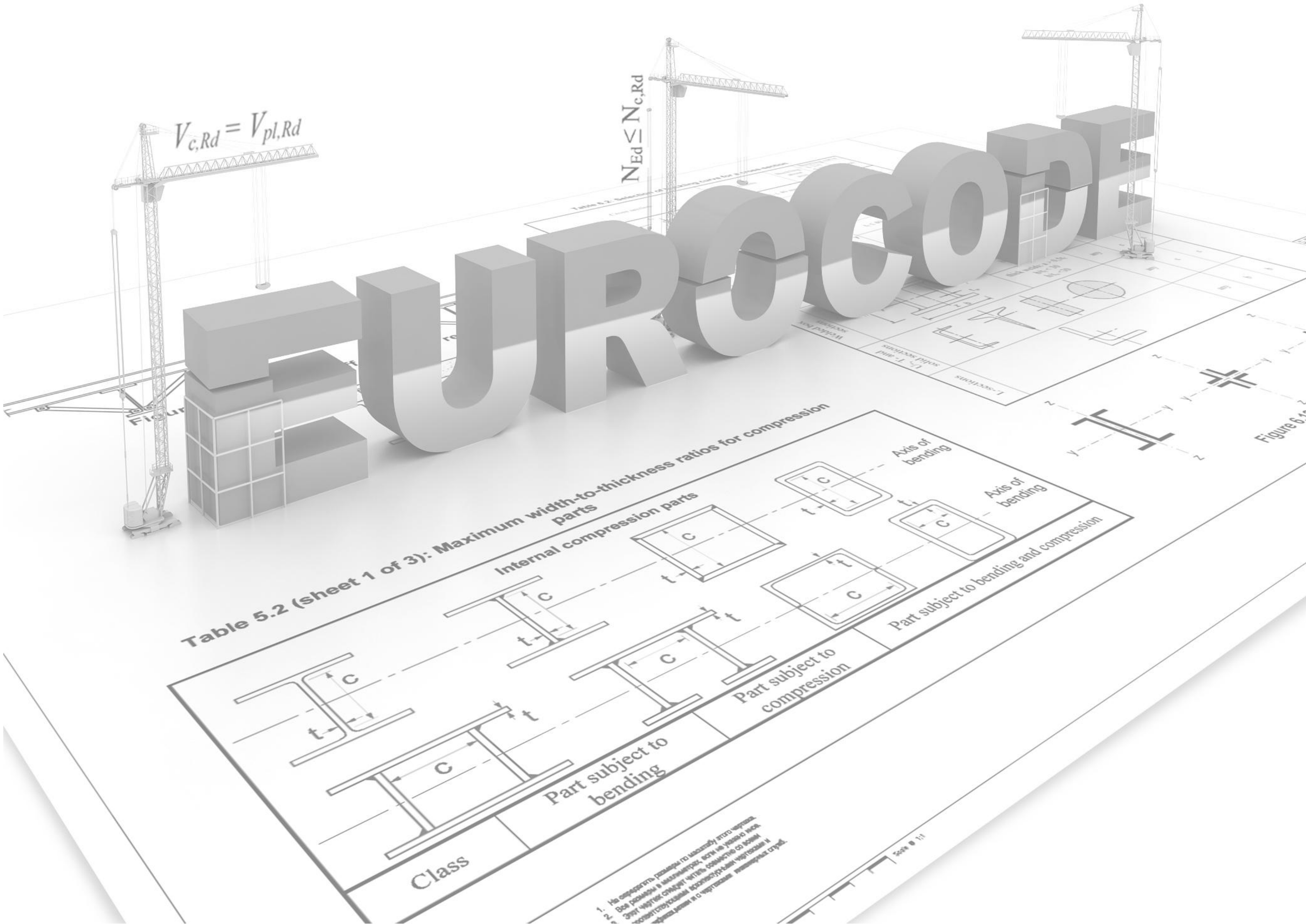
CHAPTER 4 Steel Design Tutorial

This chapter enables the user to get acquainted with the steel design procedure in midas Gen as per EN1993-1-1: 2005. It encompasses the overall design procedure, from generating load combinations to checking design results with updated sections.

CHAPTER 1

Why midas Gen

Design Examples using midas Gen to Eurocode3



Why midas Gen

01 Intuitive User Interface

The intuitive User Interface, contemporary Computer Graphics and substantially fast Solver Speed are some of the highlights of midas Gen. The user-oriented input/output features and significant analysis capabilities enable the practicing engineers and re-searchers to readily undertake structural analyses and designs for all types of buildings and even complex and long-span structures.

02 Advanced Analysis Features

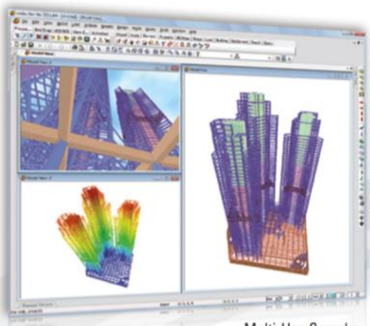
midas Gen offers conventional analysis capabilities as well as other analyses such as Geometric Nonlinear Analysis reflecting Large Displacement, Boundary Nonlinear Analysis, Pushover Analysis, Construction Simulated Analysis reflecting time dependent material properties, Heat of Hydration Analysis, etc.

03 Accurate and Practical Results

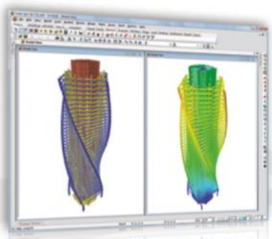
Diverse ranges of specialty finite elements in conjunction with the latest theories of structural analyses render accurate and practical results. It is prominent for providing convenience, efficiency, versatility and productivity.

04 Design Capabilities

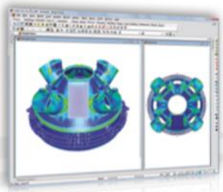
midas Gen provides design capabilities using various standards of different countries reflecting conventional as well as unusual design conditions, leading to Optimal Design. midas Gen has been used for over 20 years and applied to over an uncountable number of projects successfully, thereby, demonstrating its credibility and stability.



Multi-Use Complex

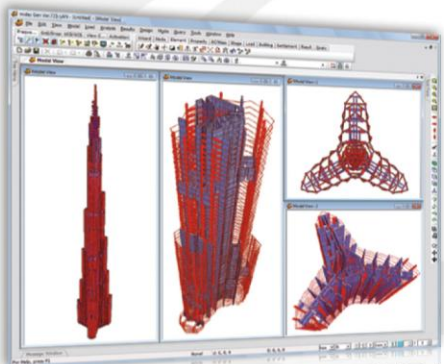


Moscow City by RMJM

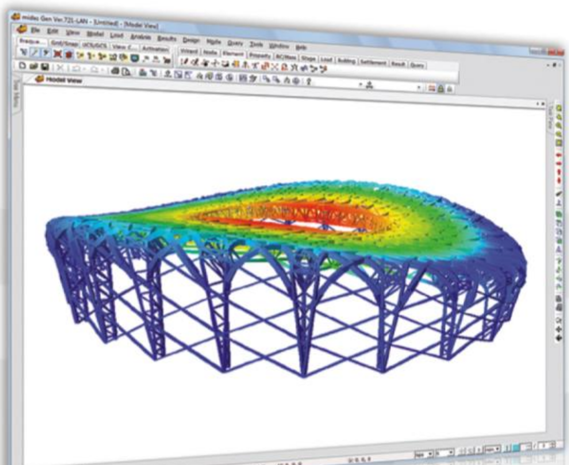


Blast Furnace (Top Shell)

midas Gen



Burj Dubai



Beijing Olympic Stadium

Features

Comprehensive Design

- **Concrete:**
  ACI318, Eurocode 2, BS8110 , IS456 & IS13920, CSA-A23.3, GB50010, AIJ-WSD, TWN-USD, AIK-USD & WSD, KSCE-USD, KCI-USD
- **Steel:**
  AISC-ASD & LRFD, AISI-CFSD, Eurocode 3, BS5950, IS800, CSA-S16, GBJ17 & GB50017, AIJ-ASD, TWN-ASD & LSD, AIK-ASD & LSD & CFSD, KSCE-ASD, KSSC-ASD
- **SRC:**
  SSRC, JGJ138, CECS28, AIJ-SRC, TWN-SRC, AIK-SRC2K & SRC, KSSC-CFT
- **Footing Design:** ACI318, BS8110

Wind & Seismic loads auto-Generation

- **Wind load:**
  IBC2000, UBC, ANSI, Eurocode 1, BS6399, IS875, NBC, GB, Japan, Taiwan & Korea
- **Seismic Load:**
  IBC2000, UBC, ATC 3-06, Eurocode 8, IS1893, NBC, GB, Japan, Taiwan & Korea

High-rise Specific Functionality

- **3-D Column Shortening** Reflecting change in modulus, creep and shrinkage
- **Construction Stage Analysis** accounting for change in geometry, supports and loadings
- Building model **generation wizard**
- Automatic mass conversion
- Material stiffness changes for **cracked sections**

High-end Analysis Capabilities

- **P-Delta** & Large displacement analysis
- Dynamic Analysis (**Time History, Response Spectrum**, etc.)
- Base Isolators & Dampers
- **Pushover** Nonlinear Analysis
- Inelastic Time History Analysis
- Staged **post-tensioning**
- Catenary Cable Structure

Intuitive User Interface

- Works Tree (Input summary with powerful modeling capabilities)
- Models created and changed with ease
- Floor Loads defined by areas and on inclined plane
- Built-in **Section property Calculator**
- **Tekla** Structures, **Revit Structure** & **STAAD** interfaces

Why midas Gen?

midas Gen is a Windows-based, **general-purpose** structural analysis and optimal design system.

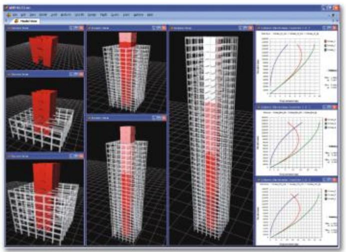
The **intuitive User Interface**, contemporary Computer Graphics and substantially fast **Solver Speed** are some of the highlights of midas Gen.

The user-oriented input/output features and significant analysis capabilities enable the **practicing engineers** and researchers to readily undertake structural **analysis and design** for even complex and large structures.

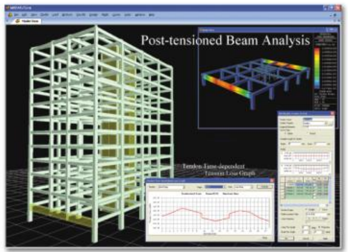
The fastest Multi-Frontal Solver and the latest analysis algorithms instantly bring accuate and practical analysis results.

In addition, midas Gen provides design capabilities using various standards of different countries leading to an Optimal Design Solution.

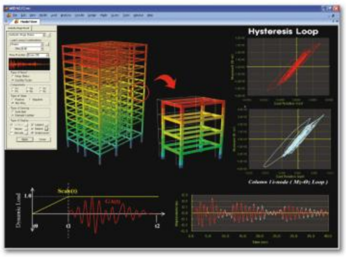
High-end Analysis Features



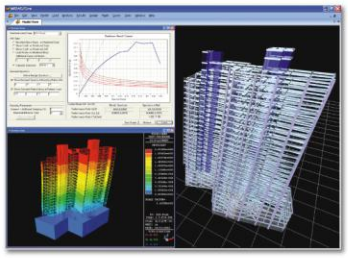
[Construction Stage Analysis]



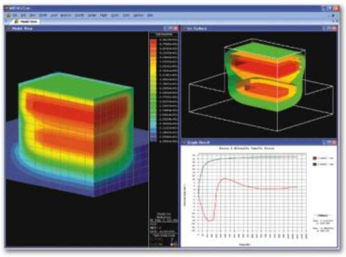
[Post-tension Analysis]



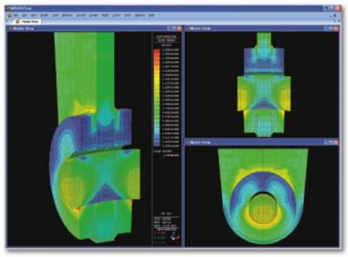
[Time History Analysis]



[Pushover Analysis]



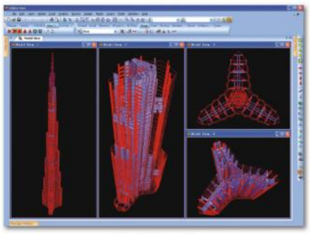
[Heat of Hydration Analysis]



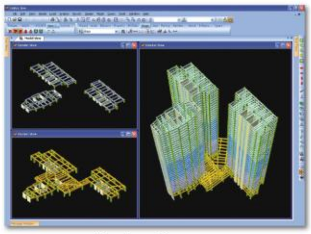
[Detail Analysis]

Project Applications

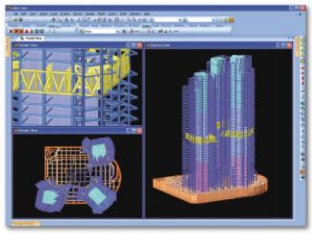
Buildings



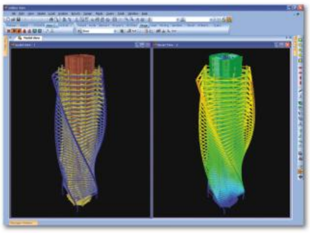
Burj Dubai



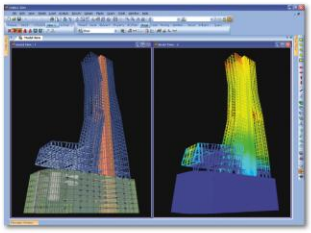
Venture Towers



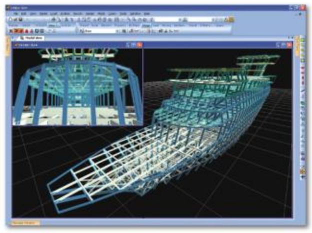
Multi-Use Complex



Moscow City by RMJM

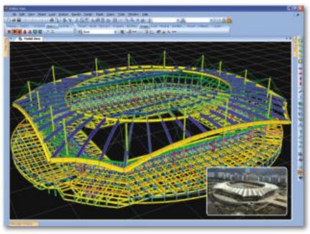


Office Building

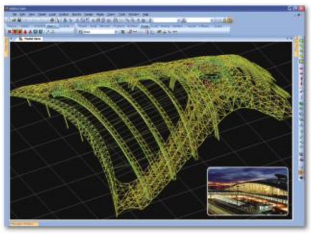


Resort Facility

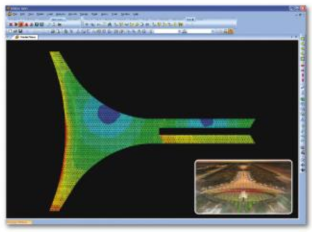
Specialty Structures



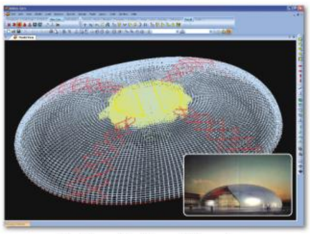
Seoul World Cup Stadium



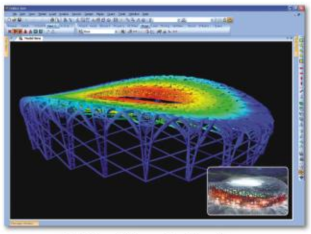
Incheon International Airport



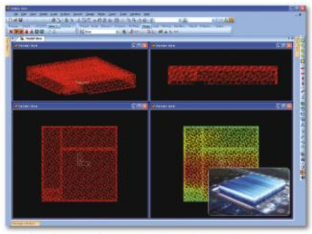
Beijing International Airport



Beijing National Theater

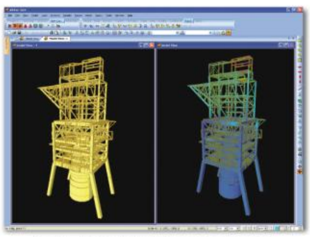


Beijing Olympic Stadium

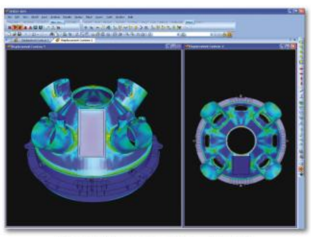


Beijing Olympic Aquatics Center

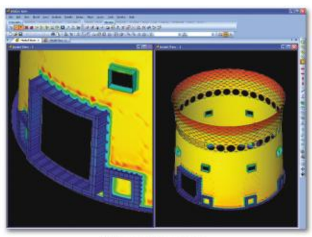
Plant Structures



India lisco blast furnace tower



Blast Furnace (Top Shell)

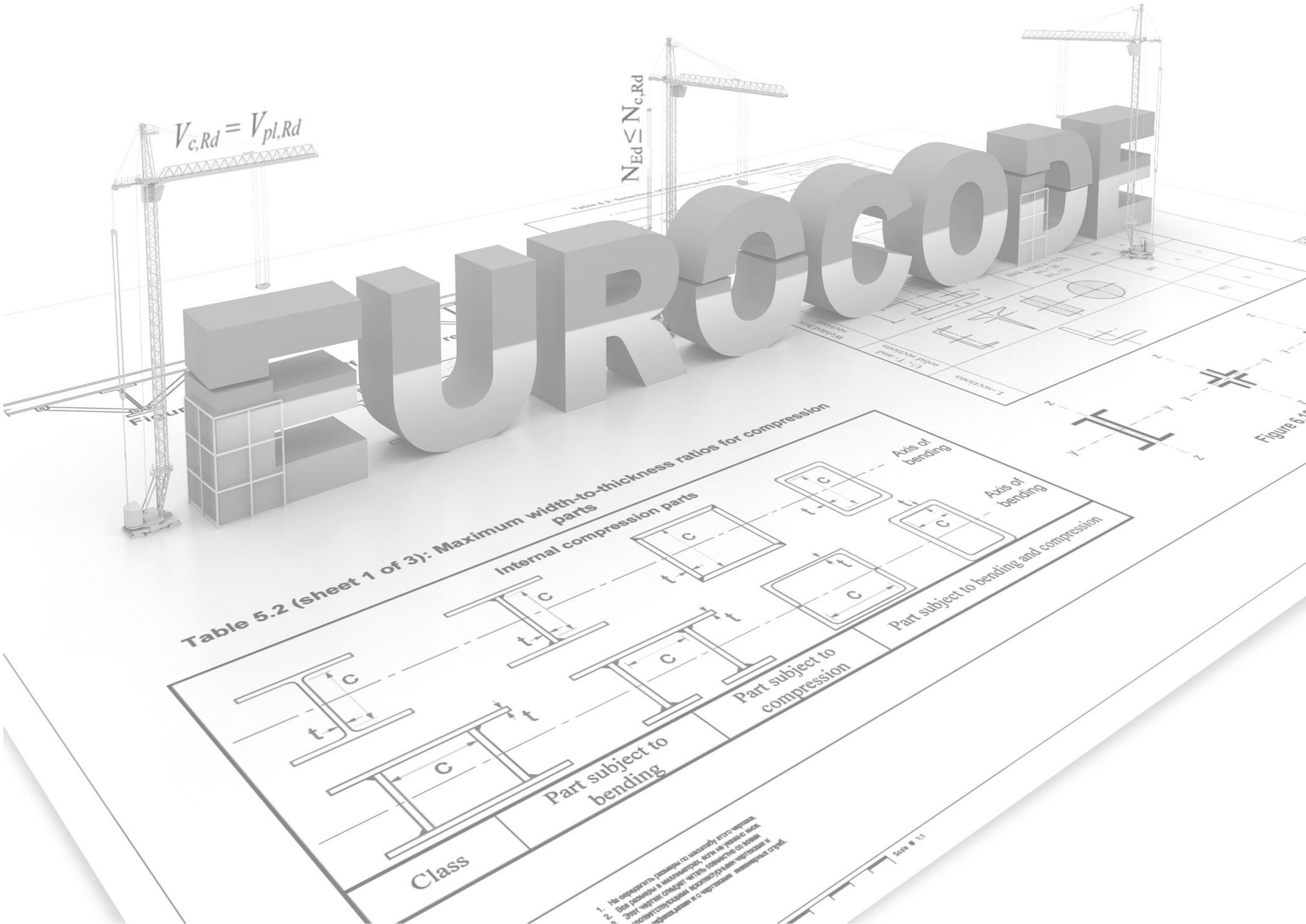


Blast Furnace

CHAPTER 2

Steel Design Algorithm

Design Examples using midas Gen to Eurocode3



CHAPTER 2

Steel Design Algorithm

as per EN1993-1-1:2005

2.1 Overview

(1) General
- Material Properties
- Section table for the application of Ultimate Limit State Check

(2) Ultimate Limit State Check
- Resistance of cross-sections
- Buckling resistance of members

(3) Serviceability Limit State Check
- Vertical deflections
- Horizontal deflections
- Dynamic effects

2.2 General

(1) Material Properties
- The nominal values of the yield strength (fy) and the ultimate strength (fu) for structural steel

Steel Grade	$t \le 40mm$		$t > 40mm$	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
S235	235	360	215	360
S275	275	430	255	410
S355	355	510	355	470
S450	440	550	410	550

- Modulus of Elasticity = 210,000 N/mm^2
- Poisson's Ratio, v, = 0.3
- Thermal Coefficient = 12 x10-6 /^{o}C
- Weight Density = 76.98 kN/m^3

(2) Section table for the application of Ultimate Limit State Check

Cross section		Limit States				
		Yielding	FB[1]	SB		LTB
				Strong axis	Weak axis	
I section	Doubly Symmetric	√	√	√	N/A	√
	Singly Symmetric	√	√	√	N/A	N/A
Box		√	√	√	√[2]	N/A
Angle		√	√	N/A	N/A	N/A
Channel		√	√	√	N/A	N/A
Tee		√	√	N/A	N/A	N/A
Double Angle		√	√	N/A	N/A	N/A
Double Channel		√	√	√	N/A	N/A
Pipe		√	√	N/A	N/A	N/A
Solid Rectangle		√	√	N/A	N/A	N/A
Solid Round		√	√	N/A	N/A	N/A
U-Rib		N/A	N/A	N/A	N/A	N/A

Note

FB: Flexural Buckling, **SB:** Shear Buckling, **LTB:** Lateral-Torsional Buckling

(1) Torsional Buckling and Torsional-Flexural Buckling are not evaluated.

(2) The thickness of two webs should be identical, and the member type should be “column” for the weak axis shear buckling check.

2.3 Ultimate Limit State Check

(1) Resistance of cross-sections

- Tension

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}}$$

- Design tension resistance
  - The design ultimate resistance of the net cross-section at holes for fasteners is not considered in midas Gen.

- Compression
  - Design compression resistance

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}}$$ For class 1,2 and 3 cross sections

$$N_{c,Rd} = \frac{A_{eff}\, f_y}{\gamma_{M0}}$$ For class 4 cross sections

  - In the case of unsymmetrical Class 4 sections, the additional moment due to the eccentricity of the centroidal axis of the effective section is considered in midas Gen.

- Bending moment
  - Design bending resistance

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl}\, f_y}{\gamma_{M0}}$$ For class 1 or 2 cross sections

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min}\ f_y}{\gamma_{M0}}$$ For class 3 cross sections

$$M_{c,Rd} = \frac{W_{eff,min}\ f_y}{\gamma_{M0}}$$ For class 4 cross sections

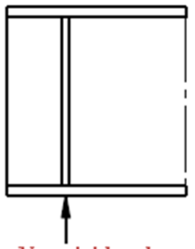
- Shear
  - Design shear resistance in the absence of torsion

$$V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}}$$

  - The shear area Av is calculated based on the clause 6.2.6 (3) as per EN1993-1-1
  - Rolled I and H sections, load parallel to web: $\mathrm{A} - 2\mathrm{bt_f} + (\mathrm{t_w} + 2\mathrm{r})\mathrm{t_f}$
  - but not less than Design elastic shear resistance is not applied.

- Shear Buckling
  - The shear buckling resistance for webs without intermediate stiffeners is calculated, according to section 5 of EN 1993-1-5, if

$$\frac{h_w}{t_w} > 72\frac{\varepsilon}{\eta} \qquad \varepsilon = \sqrt{\frac{235}{f_y\,[N/mm^2]}}$$



- For steel grades up to and including S460: η =1.20
- For higher steel grades: η =1.00
- Design resistance

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \le \frac{\eta\, f_{yw}\, h_w\, t}{\sqrt{3}\,\gamma_{M1}}$$

$$V_{bw,Rd} = \chi_w\, f_{yw}\, \frac{h_w\, t}{\sqrt{3}\gamma_{M1}}$$

$$V_{bf,Rd} = \frac{b_f\, t_f^2\, f_{yf}}{c\gamma_{M1}}\left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right)$$

- Stiffener design to resist shear buckling is not provided in midas Gen.
- Stiffener type for end supports is assumed as a non-rigid end post.
- It is assumed that the length of an unstiffened plate, 'a' is the same as the unbraced length.

• Torsion

- The torsional resistance is not checked.

• Bending and Shear

- The effect of shear force on the moment resistance is considered.
- Where the shear force is less than half the plastic shear resistance, its effect on the moment resistance is neglected.
- Where $V_{ED} \ge 0.5V_{pl,Rd}$

I-cross-sections with equal flanges and bending about the major axis

$$M_{y,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w}\right] f_y}{\gamma_{M0}}$$

but, $M_{y,V,Rd} \le M_{y,c,Rd}$

$$\rho = \left(\frac{2V_{ED}}{V_{pl,Rd}} - 1\right)^2$$

Torsion is not considered when calculating ρ

For the other cases

$$M_{V,Rd} = (1-\rho)M_{c,Rd}$$

- Bending and Axial Force
  - The effect of axial force on the moment resistance is considered.
  - Class 1 and 2 cross sections

    For doubly symmetrical I- and H-sections, allowance is not made for the effect of the axial force on the plastic resistance moment about the y-y axis when both the following criteria are satisfied:

$$N_{Ed} \leq 0.25\, N_{pl,Rd} \qquad N_{Ed} \leq \frac{0.5\, h_w\, t_w\, f_y}{\gamma_{M0}}$$

For doubly symmetrical I- and H-sections, allowance is not made for the effect of the axial force on the plastic resistance moment about the z-z axis when:

$$N_{Ed} \leq \frac{h_w\, t_w\, f_y}{\gamma_{M0}}$$

The following equations are used for standard rolled I or H sections and for welded I or H sections with equal flanges:

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

for $n \leq a$: $M_{N,z,Rd} = M_{pl,z,Rd}$

for $n > a$: $M_{N,z,Rd} = M_{pl,z,Rd}\left[1-\left(\frac{n-a}{1-a}\right)^2\right]$

Where $n = N_{Ed}/N_{pl,Rd}$

$a = (A - 2bt_f)/A$ but $a \leq 0.5$

- Bending and Axial Force

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$ for Class 1&2 sections

I and H section: α=2; β=5n but β≥1

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$ for Class 1,2,3 & 4 sections

- Bending, Shear and Axial Force
  - Where the shear force exceeds 50% of the plastic shear resistance, its effect on the moment of resistance is reflected in the formula above.
  - $M_{pl,y,Rd}$ and $M_{pl,z,Rd}$ are replaced by $M_{y,v,Rd}$ and $M_{z,v,Rd}$ respectively in the following equations to consider shear effect in the above criterion a).

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5\, a_w)$$

$$M_{N,z,Rd} = M_{pl,z,Rd}(1-n)/(1-0.5\, a_r)$$

- $M_{y,\,Rd}$ and $M_{z,\,Rd}$ are replaced by $M_{y,v,Rd}$ and $M_{z,v,Rd}$ respectively in the above criterion b) to consider shear effect.

(2) Buckling resistance of members

- Uniform members in compression

  For slenderness $\lambda \leq 0.2$ or for $\frac{N_{Ed}}{N_{cr}} \leq 0.04$ the buckling effects are ignored.

  $\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1,2 and 3 cross-sections $\quad \bar{\lambda} = \sqrt{\frac{A_{eff}\,f_y}{N_{cr}}}$ for Class 4 cross-section

  N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

$$N_{cr} = \frac{\pi^2 EI}{L_e^2}$$

- Flexural buckling is checked for the L, C, I, T, Box, Pipe, Double L, and Double C section.
- Torsional and torsional-flexural buckling is not checked.
- Design buckling resistance

$N_{b,Rd} = \frac{\chi A\,f_y}{\gamma_{M1}}$ for Class 1,2 and 3 cross-sections

$N_{b,Rd} = \frac{\chi\,A_{eff}\,f_y}{\gamma_{M1}}$ for Class 4 cross-sections

$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^{-2}}}$ but $\chi \leq 1.0$ $\quad \Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$

Buckling Curve	a_0	*a*	*b*	*c*	*d*
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

- Uniform members in bending
  - For the uniform and doubly symmetric I cross-sections only, the lateral torsional buckling check is provided.
  - It is assumed that the section is loaded through its shear center, and the boundary conditions at each end are both restrained against lateral movement and restrained against rotation about the longitudinal axis.
  - For slenderness $\bar{\lambda}_{LT} \leq \bar{\lambda}_{LT,0}$ or for $\frac{M_{ED}}{M_{cr}} \leq \bar{\lambda}_{LT,0}^2$ the lateral torsional buckling effects are ignored.

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \qquad \bar{\lambda}_{LT,0} = 0.4$$

M_{cr} is the elastic critical moment for lateral-torsional buckling. The value of C_l depends on the moment distribution along the member which is calculated based on the table in the following page

$$M_{cr} = C_l \frac{\pi^2 EI_z}{L_{cr,LT}^2} \sqrt{\frac{I_w}{I_z} + \frac{L_{cr,LT}^2 GI_t}{\pi^2 EI_z}} \qquad G = \frac{E}{2(l+v)} \qquad I_w = \frac{I_z (h-t_f)^2}{4}$$

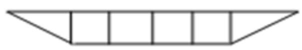
: Warping Constant

- If the member type is column, C1 is calculated based on the table below. EN 1993-1-1: 1992 Annex.

ψ	k	C1
+1	1.0	1.000
	0.7	1.000
	0.5	1.000
+3/4	1.0	1.141
	0.7	1.270
	0.5	1.305
+1/2	1.0	1.323
	0.7	1.473
	0.5	1.514
+1/4	1.0	1.563
	0.7	1.739
	0.5	1.788

ψ	k	C1
0	1.0	1.879
	0.7	2.092
	0.5	2.150
-1/4	1.0	2.281
	0.7	2.538
	0.5	2.609
-1/2	1.0	2.704
	0.7	3.009
	0.5	3.093
-3/4	1.0	2.927
	0.7	3.009
	0.5	3.093
-1	1.0	2.752
	0.7	3.063
	0.5	3.149

- If the member type is beam, C1 is calculated based on the table below.

Conditions	Bending moment diagram	*k*	*C1*
Case 1		1.0	1.132
		0.5	0.972
Class 2		1.0	1.285
		0.5	0.712
Case 3		1.0	Same as Case 1
		0.5	
Case 4		1.0	Same as Case 2
		0.5	
Case5		1.0	Same as Case 1
		0.5	

- Design buckling Resistance

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}}$$

$W_y = W_{pl,y}$ for Class 1 or 2 cross-section

$W_y = W_{el,y}$ for Class 3 cross-section

$W_y = W_{eff,y}$ for Class 1 or 2 cross-section

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{{\Phi_{LT}}^2 + {\bar{\lambda}_{LT}}^2}} \quad \text{but} \quad \chi_{LT} \le 1.0$$

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2)\right) + {\bar{\lambda}_{LT}}^2\right]$$

Buckling Curve	*a*	*b*	*c*	*d*
Imperfection factor α_{LT}	0.21	0.34	0.49	0.76

- The method in the Clause 6.3.2.3 and 6.3.2.4 of EC3 are not considered.

- Uniform members in bending and axial compression
  - For members which are subjected to combined bending and axial compression, the resistance to lateral and lateral-torsional buckling is verified by the following criteria.

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\chi_{LT}\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\chi_{LT}\frac{M_{z,Rk}}{\gamma_{M1}}} \le 1$$

K_{yy}, k_{yz}, k_{zy}, k_{zz} are the interaction factors. These values are obtained from Annex A in EN 1993-1-1: 2005. $C_{my,}$ C_{mz}, C_{mLT} in Annex A can be either user defined or auto-calculated.

Vales for $N_{Rk} = f_y A_i$, $M_{i,Rk}=f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_j	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y, Ed}$	0	0	0	e_{Ny} ,N_{Ed}
$\Delta M_{z, Ed}$	0	0	0	e_{Nz} ,N_{Ed}

- When the design axial force, NEd is larger than Ncr,z or Ncr,TF, the criteria above are not applied.
- General method of the clause 6.3.4 is not considered.

2.4 Serviceability Limit State Check

(1) Vertical Deflection

- Vertical deflection can be checked for beam member.
- Remaining total deflection (wmax) caused by the permanent and variable actions is automatically checked based on the serviceability load combinations.
- The default limit value is set to L/250
- The deflection due to the variable actions can be checked manually by adding load combination consisting of variable actions and changing the limit value

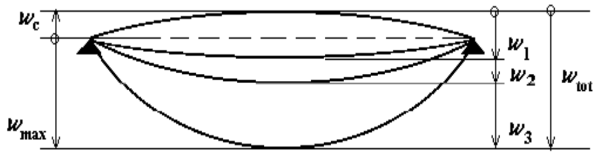


Figure A1.1 - Definitions of vertical deflections

(2) Horizontal Deflection

- Horizontal deflection can be checked for column members.
- Horizontal displacement over a story height Hi is automatically checked based on the serviceability load combinations.
- The default limit value is set to Hi/300.
- Overall horizontal displacement over the building height H should be checked separately.

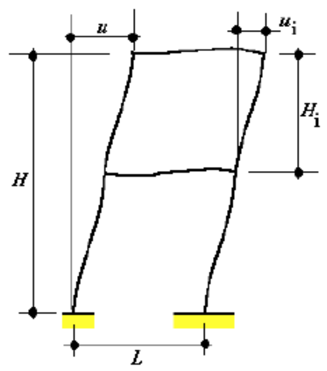


Figure A1.2 - Definition of horizontal displacements

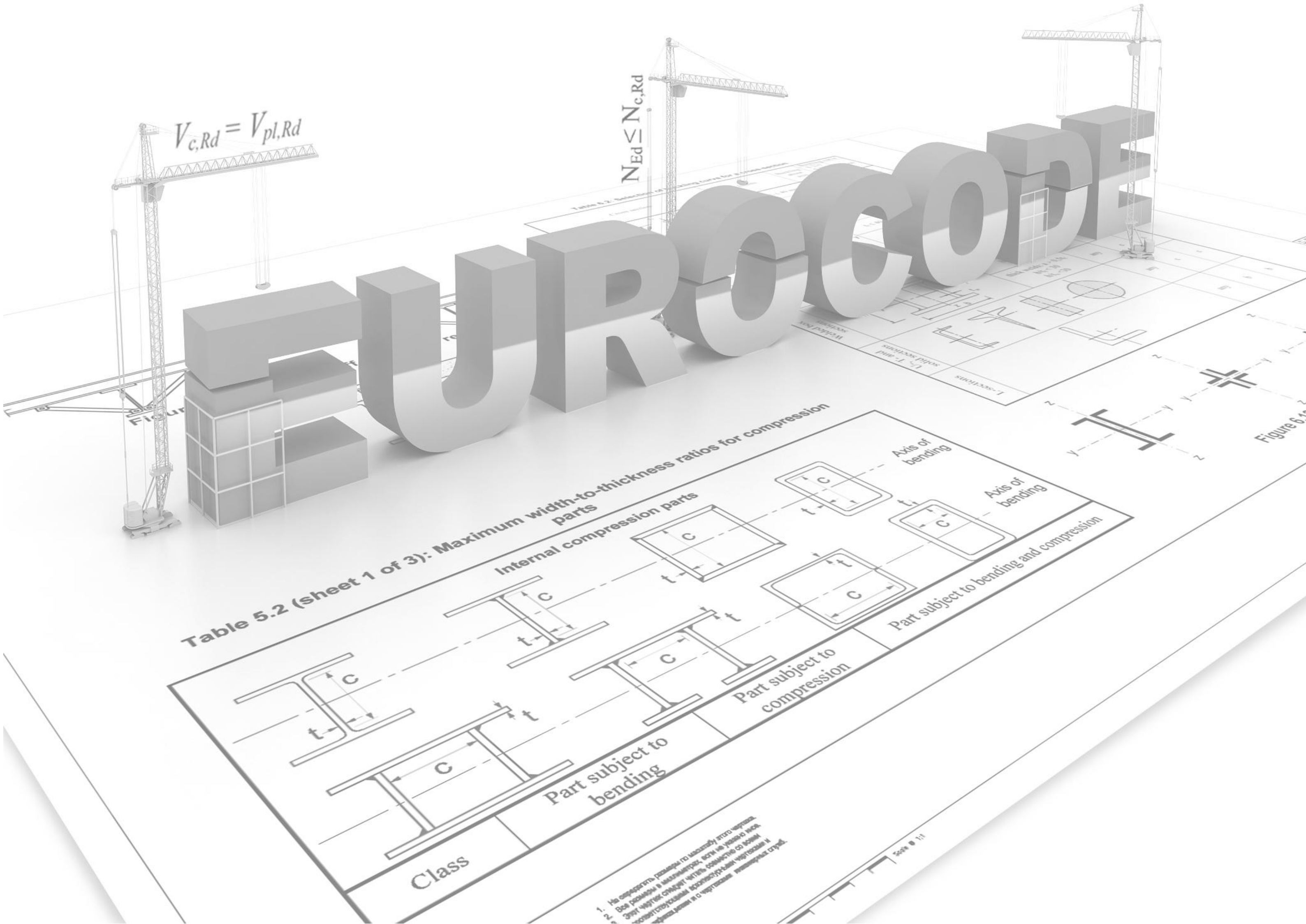
(3) Dynamic effects

- The vibration of structures is not checked.

CHAPTER 3

Verification Examples

Design Examples using midas Gen to Eurocode3

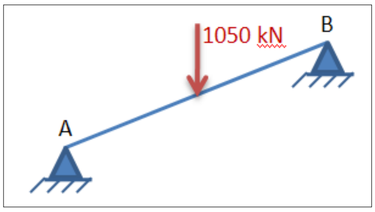


CHAPTER 3

Steel Design Verification Examples

3.1 Cross-section resistance under combined bending and shear

A short-span (1.4m), simply supported, laterally restrained beam is to be designed to carry a central point load of 1050 KN, as shown in the right figure. The arrangement of the figure results in a maximum design shear force VED of 525 KN and a maximum design bending moment MED of 367.5 kNm. In this example a 406 x 178 x 74 UB in grade S275 steel is assessed for its suitability for this application.

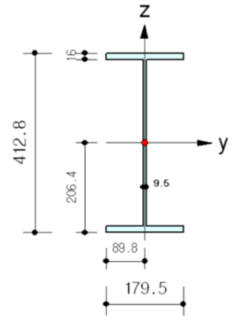


3.1.1 Material Properties

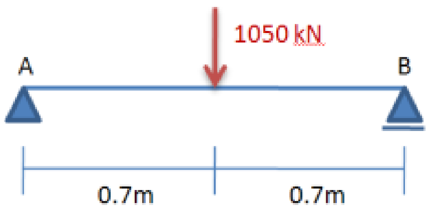
Material	S275	f_y = 275N/mm^2	E_s = 210 GPa

3.1.2 Section Properties

Section Name	406 x 178 x 74 UB
Depth *(H)*	412.8mm
Width *(B)*	179.5mm
Flange Thickness *(T_f)*	16.0 mm
Web Thickness *(T_w)*	9.5 mm
Gross sectional area *(A)*	9450.0mm^2
Shear area *(A_{sz})*	4184.0 mm^2



3.1.3 Analysis Model

Loading condition	

Beam Diagram	SF	
	BM	

3.1.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	689.25kN	689.2kN	0.01%
Bending resistance	412.50kNm	412.0kNm	0.12%
Combined resistance	386.55kNm	386.8kNm	0.06%

3.1.5 Detailed comparison

midas Gen

1. Cross-section classification

( ). Determine classification of compression outstand flanges.

[Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5]

-. e = SQRT (235/fy) = 0.92
-. b/t = BTR = 4.67
-. sigma1 = 0.278 kN/mm^2.
-. sigma2 = 0.278 kN/mm^2.
-. BTR < 9*e **(Class 1 : Plastic).**

( ). Determine classification of bending Internal Parts.

[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.92
-. d/t = HTR = 37.94
-. sigma1 = 558989.618 KPa.
-. sigma2 = -558989.618 KPa.
-. HTR < 72*e **(Class 1 : Plastic).**

2. Check Bending Moment Resistance

( ). Calculate plastic resistance moment about major axis.

[Eurocode3:05 6.1, 6.2.5]

-. Wply = 0.0015 m^3.
-. Mc_Rdy = Wply * fy / Gamma_M0 = 412.50 kN-m.

( ). Check ratio of moment resistance (M_Edy/Mc_Rdy).

```
       M_Edy        367.50
-. ------------ = ------------- = 0.891 < 1.000 ---> O.K.
       Mc_Rdy       412.50
```

3. Shear resistance of cross-section

( ). Calculate shear area.

[Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2]

-. eta = 1.2 (Fy < 460 MPa.)

Example book

1. Cross-section classification (clause 5.5.2)

$\varepsilon = \sqrt{235/f_y} = \sqrt{235/275} = 0.92$

Outstand flange in compression (Table 5.2, sheet 2):
$C = (b - t_w - 2r)/2 = 74.8$ mm
$c/t_f = 74.8/16.0 = 4.68$
Limit for Class 1 flange=9ε=8.32
8.32>4.68 ∴ flange is Class 1

Web – internal part in bending (Table 5.2, sheet 1):
$C = h - 2t_f - 2r = 360.4$ mm
$c/t_w = 360.4/9.5 = 37.94$
Limit for Class 1 web = 72ε = 66.56
66.56 > 37.94 ∴ web is Class 1

Therefore, the overall cross – section classification is Class 1.

2. Bending resistance of cross – section (clause 6.2.5)

$M_{c,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}}$ for Class 1 or 2 cross –sections

The design bending resistance of the cross-section

$M_{c,y,Rd} = \frac{275 \times 10^3 \times 275}{1.00} = 412 \times 10^6$ N mm = 412 KNm

412 KNm > 367.5 KNm

∴ cross-section resistance in bending is acceptable.

3. Shear resistance of cross-section of cross-section (clause 6.2.6)

$V_{pl,Rd} = \frac{A_v (f_y/3)}{\gamma_{M0}}$

-. r = 10.2000 mm.
-. Avy = Area - hw*tw = 5832.4000 mm^2.
-. Avz1 = eta*hw*tw = 4341.1200 mm^2.
-. Avz2 = Area - 2*B*tf + (tw + 2*r)*tf = 4184.4000 mm^2.
-. Avz = MAX[Avz1, Avz2] = 4341.1200 mm^2.

( ). Calculate plastic shear resistance in local-z direction (Vpl_Rdz). [Eurocode3:05 6.1, 6.2.6]
-. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0 = 689.25 kN.

( ). Shear Buckling Check. [Eurocode3:05 6.2.6]
-. HTR < 72*e/Eta ---> No need to check!

( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).
(LCB = 1, POS = J)
-. Applied shear force : V_Edz = 525.00 kN.

```
      V_Edz        525.00
-. ------------- = ----------- = 0.762 < 1.000 ---> O.K.
      Vpl_Rdz      689.25
```

4. Check Interaction of Combined Resistance

( ). Calculate Major reduced design resistance of bending and shear. [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edz / Vpl_Rdz > 0.5 (equal flanges)
-. Rho = { 2*(V_Edz/Vpl_Rdz) - 1 }^2 = 0.274
-. My.V_Rd1= [Wply - {Rho*Aw^2/(4*tw)}]*fy / Gamma_M0 = 386.55 kN-m.
-. My_Rd = MIN [My.V_Rdy1, Mc_Rdy] = 386.55 kN-m.

( ). Calculate Minor reduced design resistance of benging and shear. [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edy / Vpl_Rdy < 0.5
-. Mz_Rd = Mc_Rdz = 73.42 kN-m.

( ). Check general interaction ratio.
[Eurocode3:05 6.2.1 (6.2)] - Class1 or Class2

```
                N_Ed      M_Edy     M_Edz
-. Rmax1 = --------- + ----------- + ---------
                N_Rd      My_Rd     Mz_Rd
        = 0.951 < 1.000 ---> O.K.
```

( ). Check interaction ratio of bending and axial force member.
[Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2
-. n = N_Ed / Npl_Rd = 0.000
-. a = MIN[(Area-2b*tf)/Area, 0.5] = 0.392
-. Alpha = 2.000
-. Beta = MAX[5*n, 1.0] = 1.000

For a rolled I section, loaded parallel to the web, the shear area A_v is given b y

$A_v = A - 2bt_f + (t_w + r)\, t_f$ (bur not less than $\eta h_w t_w$)

η= 1.2 (from EN 1993-1-5, though the UK National Annex may specify an alternative value).

$h_w = (h - 2t_f) = 412.8 - (2 \times 16.0) = 380.8$ mm

$\therefore\ A_v = 9450 - (2 \times 179.5 \times 16.0) + (9.5 + 10.2) \times 16.0 = 4184\ \text{mm}^2$

(but not less than $1.2 \times 380.8 \times 9.5 = 4341\ \text{mm}^2$)

$$V_{p1,Rd} = \frac{4341 \times (275/3)}{1.00} = 689200\ \text{N} = 689.2\ \text{KN}$$

Shear buckling need not be considered, provided

$\frac{h_w}{t_w} \le 72\frac{\varepsilon}{\eta}$ for unstiffened webs

$$72\frac{\varepsilon}{\eta} = 72 \times \frac{0.92}{1.2} = 55.5$$

Actual h_w/t_w= 380.9/9.5 = 40.1

40.1 ≤ 55.5 ∴ no shear buckling check required

689.2 > 525 KN ∴ shear resistance is acceptable

4. Resistance of cross-section to combined bending and shear (clause 6.2.8)

The applied shear force is greater than half the plastic shear resistance of the cross-section, therefore a reduced moment resistance $M_{y,V,Rd}$ must be calculated. For an I section (with equal flanges) and bending about the major axis, clause 6.2.8(5) and equation (6.30) may be utilized.

$$M_{y,V,Rd} = \frac{(W_{pl,y} - \rho A_w^2/4t_w)f_y}{\gamma_{M0}} \quad \text{but } M_{y,V,Rd} \le M_{y,c,Rd}$$

$$\rho = \left(\frac{2V_{ED}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2 \times 525}{689.2} - 1\right)^2 = 0.27$$

$A_w = h_w t_w = 380.8 \times 9.5 = 3617.6\ \text{mm}^2$

$$\Rightarrow M_{y,V,Rd} = \frac{1501000 - 0.27 \times 3617.6^2/4 \times 9.5) \times 275}{1.0} = \mathbf{386.8\ KN}$$

386.8 KNm > 376.5 KNm

∴ cross-section resistance to combined bending and shear is acceptable

Conclusion

A 406 × 178 × 74 UB in grade S275 steel is suitable for the arrangement and loading shown by Fig. 6.13

-. N_Ed < 0.25*Npl_Rd = 649.69 kN.
-. N_Ed < 0.5*hw*tw*fy/Gamma_M0 = 497.42 kN.
Therefore, No allowance for the effect of axial force.
-. Mny_Rd = Mply_Rd = **386.55 kN-m.**
-. Rmaxy = M_Edy / Mny_Rd = 0.951 < 1.000 ---> O.K.

-. N_Ed < hw*tw*fy/Gamma_M0 = 1675.52 kN.
Therefore, No allowance for the effect of axial force.
-. Mnz_Rd = Mplz_Rd = 73.42 kN-m.
-. Rmaxz = M_Edz / Mnz_Rd = 0.000 < 1.000 ---> O.K.

-. Rmax2 = MAX[Rmaxy, Rmaxz] = 0.951 < 1.000 ---> O.K.

[Reference]
L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 53-55 (Example 6.5)

3.2 Cross-section resistance under combined bending and compression

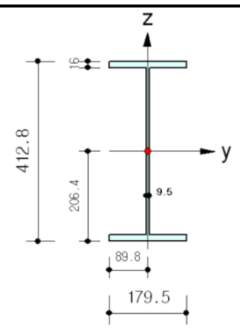
A member is to be designed to carry a combined major axis moment and an axial force. In this example, a cross-sectional check is performed to determine the maximum bending moment that can be carried by a 457 × 191 × 98 UB in grade S235 steel, in the presence of an axial force of 1400 KN.

3.2.1 Material Properties

Material	S275	f_y = 275N/mm^2	E_s = 210 GPa

3.2.2 Section Properties

Section Name	406 x 178 x 74 UB
Depth *(H)*	412.8mm
Width *(B)*	179.5mm
Flange Thickness *(T_f)*	16.0 mm
Web Thickness *(T_w)*	9.5 mm
Gross sectional area *(A)*	9450.0mm^2
Shear area *(A_{sz})*	4184.0 mm^2



3.2.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	689.25kN	689.2kN	0.01%
Bending resistance	412.50kNm	412.0kNm	0.12%
Combined resistance	386.55kNm	386.8kNm	0.06%

3.2.4 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 1.00 -. b/t = BTR = 4.11 -. sigma1 = 112000.000 KPa. -. sigma2 = 112000.000 KPa. -. BTR < 9*e (**Class 1: Plastic).** **( ). Determine classification of compression Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 1.00	***1. Cross-section classification under pure compression (clause 5.5.2)*** $\varepsilon = \sqrt{235/f_y} = \sqrt{235/235} = 1.00$ Outstand flanges (Table 5.2, sheet 2): C =(b - t_w – 2r)/2 = 80.5 mm c/t_f = 80.5/19.6 =4.11 Limit for Class 1 flange=9ε=9.0 9.0>4.11 ∴ **flange is Class 1** Web – internal part in bending (Table 5.2, sheet 1): C = h - 2t_f – 2r = 407.6 mm c/t_w = 407.6/11.4 = 35.75

-. d/t = HTR = 35.75
-. sigma1 = 112000.000 KPa.
-. sigma2 = 112000.000 KPa.
-. HTR < 38*e **(Class 2 : Compact).**

2. Check Axial and Bending Resistance

( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]
-. Kl/i = 32.3 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).
[Eurocode3:05 6.1, 6.2.4]
-. Nc_Rd = fy * Area / Gamma_M0
= **2937.50 kN**

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).

```
    N_Ed        1400.00
-. ---------- = ------------ = 0.477 < 1.000 ---> O.K.
    Nc_Rd       2937.50
```

( ). Calculate plastic resistance moment about major axis.
[Eurocode3:05 6.1, 6.2.5]
-. Wply = 0.0022 m^3.
*-. Mc_Rdy = Wply * fy / Gamma_M0 = 524.05 kN-m.*

-. N_Ed > 0.25*Npl_Rd = 695.92 kN.
-. N_Ed > 0.5*hw*tw*fy/Gamma_M0 = **573.31 kN.**
Therefore, Allowance for the effect of axial force.

3. Check Interaction of Combined Resistance

( ). Calculate Major reduced design resistance of benging and shear. [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edz / Vpl_Rdz < 0.5
-. My_Rd = Mc_Rdy = **524.05 kN-m**.

( ). Calculate Minor reduced design resistance of benging and shear. [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edy / Vpl_Rdy < 0.5
-. Mz_Rd = Mc_Rdz = 89.06 kN-m.

( ). Check interaction ratio of bending and axial force member.
[Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2
-. n = N_Ed / Npl_Rd = 0.477
-. a = MIN[(Area-2b*tf)/Area, 0.5] = 0.395
-. Alpha = 2.000
-. Beta = MAX[5*n, 1.0] = 2.383

-. N_Ed > 0.25*Npl_Rd = 695.92 kN.
-. N_Ed > 0.5*hw*tw*fy/Gamma_M0 = 573.31 kN.
Therefore, Allowance for the effect of axial force.
-. Mny_Rd = MIN[Mply_Rd*(1-n)/(1-0.5*a), Mply_Rd]
= **341.88 kN-m**.

Limit for Class 2 web = 38ε = 38.0
38.0 > 35.75 ∴ **web is Class 2**
Under pure compression, the overall cross-section classification is therefore Class 2.

2. Bending and axial force (clause 6.2.9.1)

No reduction to the plastic resistance moment due to the effect of axial force is required when both of the following criteria are satisfied.

$N_{Ed} \le 0.25 N_{pi, Rd}$

And

$N_{Ed} = \frac{0.5 h_w t_w f_y}{\gamma_{M0}}$

N_{Ed} = 1400 KN

$N_{pl, Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{12500 \times 235}{1.0}$ = **2937.5 KN**

0.25 $N_{pl, Rd}$ = 733.9 KN

733.9 KN < 1400 KN ∴ equation (6.33) is not satisfied

$\frac{0.5 h_w t_w f_y}{\gamma_{M0}} = \frac{0.5 \times [467.2-(2 \times 19.6)] \times 11.4 \times 235}{1.0}$ = **573.3 KN**

573.3 KN < 1400 KN ∴ equation (6.34) is not satisfied

Therefore, allowance for the effect of axial force on the plastic moment resistance of the cross-section must made.

3. Reduced plastic moment resistance (clause 6.2.9.I(5))

$M_{N,y, Rd} = M_{pl, y, Rd}\frac{1-n}{1-0.5a}$ but $M_{N,y, Rd} \le M_{pl, y, Rd}$

Where

$n = N_{Ed}/ M_{pl, y, Rd} = 1400/2937.5 = 0.48$

$a = (A - 2bt_f)/A = [12500 - (2 \times 192.8 \times 19.6)]/12500$
$= 0.40$

$M_{pl, y, Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{2232000 \times 235}{1.0}$ = **524.5 KNm**

$\Rightarrow M_{N,y, Rd} = 524.5 \times \frac{1-0.48}{1-(0.5 \times 0.40)}$ = **342.2 KNm**

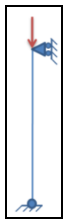
Conclusion

In order to satisfy the cross-sectional checks of clause 6.2.9, the maximum bending moment that can be carried by a 457 × 191 × 98 UB in grade S235 steel, in the presence of an axial force 1400 KN is 342.2 KNm.

[Reference]
L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 57-59 (Example 6.6)

3.3 Buckling resistance of a compression member

A circular hollow section (CHS) member is to be used as an internal column in a multi-storey building. The column has pinned boundary conditions at each end, and the inter-storey height is 4m, as shown in the right figure. The critical combination of actions results in a design axial force of 1630 KN. Assess the suitability of a hot-rolled 244.5 x 10 CHS in grade S275 steel for this application.

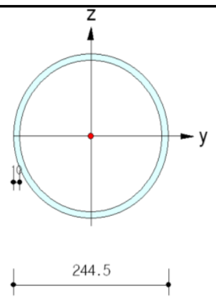


3.3.1 Material Properties

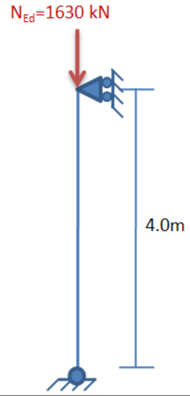
Material	S275	f_y = 275N/mm^2	E_s = 210 GPa

3.3.2 Section Properties

Section Name	244.5 X 10 CHS
Thickness *(T)*	10.0 mm
Gross sectional area (*A*)	7370 mm^2
Modulus of Elasticity ($W_{el,y}$)	415 000 mm^3
Modulus of Elasticity ($W_{pl,y)}$	550 000 mm^3
Moment of Inertia (*I*)	50 750 000 mm^4



3.3.3 Analysis Model

Loading condition	

3.3.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	2026.75 kN	2026.8 kN	0.00%
Bending resistance	1836.70 kNm	1836.5 kNm	0.06%

3.3.5 Detailed comparison

midas Gen

1. Class of Cross Section

( ). Determine classification of tublar section(hollow pipe).

[Eurocode3:05 Table 5.2 (Sheet 3 of 3)]

-. e = SQRT(235/fy) = 0.92

-. d/t = DTR = 24.45

-. DTR < 50*e^2 (Class 1 : Plastic).

2. Check Axial Resistance

( ). Check slenderness ratio of axial compression member (Kl/i) [Eurocode3:05 6.3.1]

-. Kl/i = 48.2 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).

[Eurocode3:05 6.1, 6.2.4]

-. Nc_Rd = fy * Area / Gamma_M0 = **2026.75 kN.**

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).

```
     N_Ed       1630.00
-. --------- = ------------ = 0.804 < 1.000 ---> O.K.
     Nc_Rd      2026.75
```

( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz). [Eurocode3:05 6.3.1.1, 6.3.1.2]

-. Beta_A = Aeff / Area = 1.000

-. Lambda1 = Pi * SQRT(Es/fy) = 86.815

-. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = 0.555

-. Ncry = Pi^2*Es*Ryy / KLy^2= **6571.49 kN.**

-. Lambda_by > 0.2 and N_Ed/Ncry > 0.04
--> Need to check.

-. Alphay = 0.210

-. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = **0.691**

-. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = 0.906

-. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1 = **1836.70 kN.**

-. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A) = 0.555

-. Ncrz = Pi^2*Es*Rzz / KLz^2 = 6571.49 kN.

-. Lambda_bz > 0.2 and N_Ed/Ncrz > 0.04 --> Need to check.

-. Alphaz = 0.210

-. Phiz = 0.5 * [1 + Alphaz*(Lambda_bz-0.2) + Lambda_bz^2] = **0.691**

-. Xiz = MIN [1 / [Phiz + SQRT(Phiz^2 - Lambda_bz^2)], 1.0] = 0.906

-. Nb_Rdz = Xiz*Beta_A*Area*fy / Gamma_M1 = **1836.70 kN.**

Example book

1. Cross-section classification (clause 5.5.2)

$\varepsilon = \sqrt{235/f_y} = \sqrt{235/275} = 0.92$

Tubular sections (Table 5.2, sheet 3):

d/t = 244.5/10.0 =24.5

Limit for Class 1 section =50 ε^2=42.7

42.7 > 24.5 ∴ section is Class 1

2. Cross Section Compression resistance (clause 6.2.4)

$N_{c,\,Rd} = \frac{A f_y}{\gamma_{M0}}$ for Class 1,2 or 3 cross-sections

∴ $N_{c,\,Rd} = \frac{7370 \times 275}{1.00} = 2026.8 \times 10^3$ N = **2026.8 KN**

2026.8 > 1630 KN

∴ cross-section resistance is acceptable

3. Member Buckling resistance in compression (clause 6.3.1)

$N_{b,\,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ for Class 1,2 or 3 cross-sections

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}$ but $\chi \le 1.0$

where

$\Phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$

and

$\lambda = \sqrt{\frac{A f_y}{N_{cr}}}$ for Class 1,2 or 3 cross-sections

Elastic critical force and non-dimensional slenderness for flexural buckling

$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \times 210000 \times 50730000}{4000^2}$ = **6571 KN**

∴ $\lambda = \sqrt{\frac{7370 \times 275}{6571 \times 10^3}} = 0.56$

Selection of buckling curve and imperfection factor α

For a hot-rolled CHS, use buckling curve a (Table 6.5 (Table 6.2 of EN 1993-1-)).

For curve buckling curve a, α = 0.21 (Table 6.4 (Table 6.1 of EN 1993-1-1)).

Buckling curves

$\phi = 0.5[1 + 0.21 \times (0.56 - 0.2) + 0.56^2]$ = **0.69**

$\chi = \frac{1}{0.69 + \sqrt{0.69 - 0.56^2}}$

$N_{b,\,Rd} = \frac{0.91 \times 7370 \times 275}{1.0} = 1836.5 \times 10^3$ N = **1836.5 KN**

1836.5 > 1630 KN ∴ buckling resistance is acceptable

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd).

```
 -. Nb_Rd  = MIN[ Nb_Rdy, Nb_Rdz ] =  1836.70 kN.
     N_Ed        1630.00
 -. ---------  =  -------------  =  0.887  <  1.000  ---> O.K.
    Nb_Rd       1836.70
```

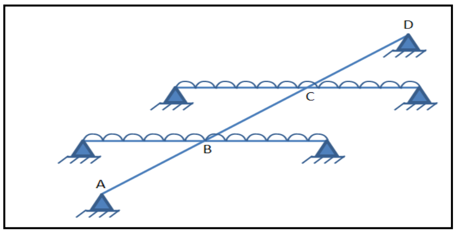
Conclusion

The chosen cross-section, 244.5 × 10 CHS, in grade S275 steel is acceptable.

[Reference]
L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 66-68 (Example 6.7)

3.4 I-section beam design under shear force and bending moment

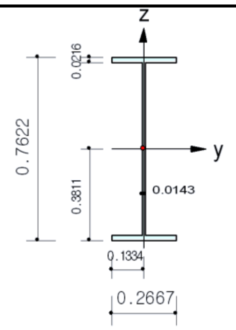
A simply supported primary beam is required to span 10.8m and to support two secondary beams as shown in Fig.6.24. The secondary beams are connected through pin plates to the web of the primary beam, and full lateral restraint may be assumed at these points. Select a suitable member for the primary beam assuming grade S275 steel.



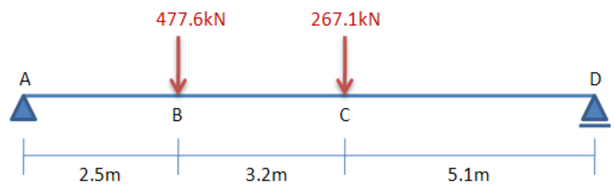
3.4.1 Material Properties

Material	S275	f_y = 275N/mm^2	E_s = 210 GPa

3.4.2 Section Properties

Section Name	762 X 267 X 173 UB	
Depth *(H)*	762.2 mm	
Width *(B)*	266.7 mm	
Flange Thickness *(T_f)*	21.6 mm	
Web Thickness *(T_w)*	14.3 mm	
Gross sectional area *(A)*	22 000mm^2	
Shear area *(A_{sz})*	11 500.2 mm^2	

3.4.3 Analysis Model

Loading condition		
Beam Diagram	SF	
	BM	

3.4.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	1958.93kN	1959.00kN	0.00%
Bending resistance	1705.00kNm	1704.00kNm	0.06%
Combined resistance	1511.41kN	1469.00kN	2.81%

3.4.5 Detailed comparison

<table>
<tr><th>midas Gen</th><th>Example book</th></tr>
<tr><td>

1. Cross-section classification

( ). compression outstand flanges (Flange)

-. e = SQRT(235/fy) = 0.92
-. b/t = BTR = 5.08
-. sigma1 = 228745.362 KPa.
-. sigma2 = 228745.362 KPa.
-. BTR < 9*e **(Class 1 : Plastic).**

( ). bending Internal Parts (Web)

-. e = SQRT(235/fy) = 0.92
-. d/t = HTR = 47.97
-. sigma1 = 588373.896 KPa.
-. sigma2 = -588373.896 KPa.
-. HTR < 72*e **(Class 1 : Plastic).**

2. Shear resistance of cross-section

( ). Calculate shear area.

[Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2]

-. eta = 1.2 (Fy < 460 MPa.)
-. r = 0.0165 m.
-. Avy = Area - hw*tw = 0.0117 m^2.
-. Avz1 = eta*hw*tw = 0.0123 m^2.
-. Avz2 = Area - 2*B*tf + (tw + 2*r)*tf
= 0.0115 m^2.
-. Avz = MAX[Avz1, Avz2] = 0.0123 m^2.

( ). Plastic shear resistance (Vpl_Rdz)

[Eurocode3:05 6.1, 6.2.6]

-. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0
= **1958.93 kN.**
-. Avz = 1.23380e-002
-. Fy = 2.75000e+005
-. Gamma_M0 = 1.00

( ). Shear Buckling Check [Eurocode3:05 6.2.6]

-. HTR < 72*e/Eta ---> No need to check!
-. e = SQRT(235/fy) = 0.92
-. d/t = HTR = 47.97

</td><td>

1. Cross-section classification

$\varepsilon = \sqrt{235/f_y} = \sqrt{235/275} = 0.92$

Outstand flanges (Table 5.2, sheet 2):
$C = (b - t_w - 2r)/2 = 109.7$ mm
$c/t_f = 109.7/21.6 = 5.08$
Limit for Class 1 flange=9ε= 8.32
8.32 > 5.08 **∴flange is Class 1**

Web – internal part in bending (Table 5.2, sheet 1):
$C = h - 2t_f - 2r = 686.0$ mm
$c/t_w = 686.0/14.3 = 48.0$
Limit for Class 1 web = 72ε = 66.6
66.6 > 48.0 **∴ web is Class 1**

2. Shear resistance of cross-section

$V_{p1,Rd} = \frac{A_v(f_y/3)}{\gamma_{M0}}$

For a rolled I section, loaded parallel to the web, the shear area A_v, is given by
$A_v = A - 2bt_f + (t_w + r)\, t_f$ (but not less than $\eta h_w t_w$)
η= 1.2 (from Eurocode 3 –part 1.5, though the UK National Annex may specify an alternative value).

$h_w = (h - 2t_f) = 762.2 - (2 \times 21.6) = 719.0$ mm
$\therefore A_v = 22000 - (2 \times 266.7 \times 21.6) + (14.3 + 16.5) \times 21.6$
$= 9813\ \text{mm}^2$
(but not less than $1.2 \times 719.0 \times 14.3 = 12338\text{mm}^2$)

$V_{p1,Rd} = \frac{12338 \times (275/3)}{1.00}$ = 1959000 N = **1959 KN**

Shear buckling need not be considered, provided
$\frac{h_w}{t_w} \leq 72\frac{\varepsilon}{\eta}$ for unstiffened webs
$72\frac{\varepsilon}{\eta} = 72 \times \frac{0.92}{1.2} = 55.5$
Actual h_w/t_w= 719.0/14.3 = 50.3
50.3 ≤ 55.5 ∴ no shear buckling check required

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

( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).
(LCB = 1, POS = J)
-. Applied shear force : V_Edz = 493.17 kN.
V_Edz 493.17
-. ---------- = ------------ = 0.252 < 1.000 ---> **O.K.**
Vpl_Rdz 1958.93

3. Bending resistance of cross-section

( ). Plastic resistance moment about major axis.
[Eurocode3:05 6.1, 6.2.5]
-. Wply = 0.0062 m^3.
-. Mc_Rdy = Wply * fy / Gamma_M0
= **1705.00 kN-m**.

( ). Ratio of moment resistance (M_Edy/Mc_Rdy).
M_Edy 1232.94
-. ---------- = ------------- = 0.723 < 1.000 ---> **O.K.**
Mc_Rdy 1705.00

( ). Plastic resistance moment about minor axis.
[Eurocode3:05 6.1, 6.2.5]
-. Wplz = 0.0008 m^3.
-. Mc_Rdz = Wplz * fy / Gamma_M0
= 221.92 kN-m.

( ). Ratio of moment resistance (M_Edz/Mc_Rdz).
M_Edz 0.00
-. ---------- = ------------- = 0.000 < 1.000 ---> O.K.
Mc_Rdz 221.92

4. Combined bending and shear resistance

( ). Major reduced design resistance of bending and shear [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edz / Vpl_Rdz < 0.5
-. My_Rd = Mc_Rdy = 1705.00 kN-m.

( ). Minor reduced design resistance of bending and shear [Eurocode3:05 6.2.8 (6.30)]
-. In case of V_Edy / Vpl_Rdy < 0.5
-. Mz_Rd = Mc_Rdz = 221.92 kN-m.

( ). General interaction ratio [Eurocode3:05 6.2.1 (6.2)] - Class1 or Class2
N_Ed M_Edy M_Edz
-. Rmax1 = -------- + --------- + ----------
N_Rd My_Rd Mz_Rd
= 0.723 < 1.000 ---> O.K.

( ). Interaction ratio of bending and axial force member [Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2
-. Alpha = 2.000
-. Beta = MAX[5*n, 1.0] = 1.000

1959 > 493.2 KN
∴ shear resistance is acceptable

3. Bending resistance of cross-section

$M_{c,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}}$ for Class 1 or 2 cross-sections

EN 1993-1-1 recommends a numerical value of γ_{M0} = 1.00 (through for buildings to be constructed in the UK, reference should be made to the National Annex). The design bending resistance of the cross-section

$$M_{c,y,Rd} = \frac{6198 \times 10^3 \times 275}{1.00} = 1704 \times 10^6 \text{ Nmm}$$

= **1704 kNm**

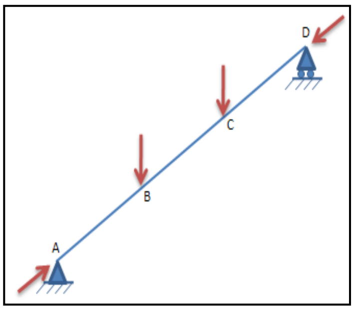
1704 KNm > 1362 KNm
∴ cross-section resistance in bending is acceptable

4. Combined bending and shear resistance

Clause 6.2.8 states that provided the shear force V_{Ed} is less than half the plastic shear resistance $V_{pl,Rd}$ its effect on the moment resistance may be neglected except where shear buckling reduces the section resistance. In this case, there is no reduction for shear buckling (see above), and maximum shear force (V_{Ed}=493.2kN) is less than half the plastic shear resistance ($V_{pl,Rd}$=1959kN). Therefore, resistance under combined bending and shear is **acceptable**.

-. n = N_Ed / Npl_Rd = 0.000 -. a = MIN[(Area-2b*tf)/Area, 0.5] = 0.476 -. Mny_Rd = MIN[Mply_Rd*(1-n)/(1-0.5*a), Mply_Rd] = 1705.00 kN-m. -. Rmaxy = M_Edy / Mny_Rd = 0.723 < 1.0 ---> O.K. -. In case of n < a -. Mnz_Rd = Mplz_Rd = 221.92 kN-m. -. Rmaxz = M_Edz / Mnz_Rd = 0.0 < 1.0 ---> O.K. -. Rmax2 = max[Rmaxy, Rmaxz] = 0.723 < 1.0 --> O.K. -. Rmax = MAX[Rmax1, Rmax2] = 0.723 < 1.0 -->**O.K**	[Reference] L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 74-79 (Example 6.8)

3.5 Member resistance under combined major axis bending and axial compression



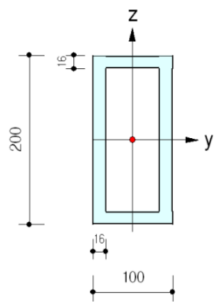
A rectangular hollow section (RHS) member is to be used as a primary floor beam of 7.2 m span in a multi-storey building. Two design point loads of 58 KN are applied to the primary beam (at locations B and C) from secondary beams, as shown in the right figure. The secondary beams are connected through fin plates to the webs of the primary beam, and full lateral and torsional restraint may be assumed at these points. The primary beam is also subjected to a design axial force of 90 KN.
Assess the suitability of a hot-rolled 200 X 100 X 16 RHS in grade S355 steel for this application. In this example the interaction factors kij (for member checks under combined bending and axial compression) will be determined using alternative method 1 (Annex A)

3.5.1 Material Properties

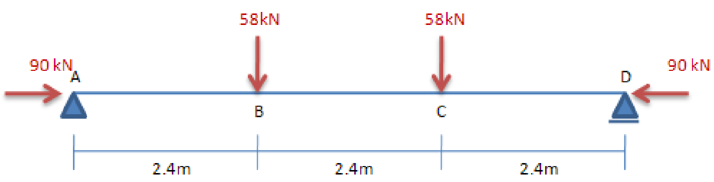
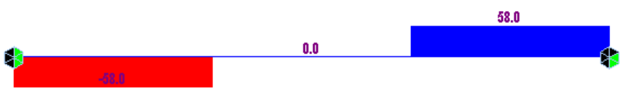
Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

3.5.2 Section Properties

Section Name	200 X 100 X 16 RHS
Depth *(H)*	200.0 mm
Width *(B)*	100.0 mm
Flange Thickness *(T_f)*	16 .0 mm
Web Thickness *(T_w)*	16.0 mm
Gross sectional area *(A)*	8300 .0 mm^2
Shear area *(A_{sz})*	5533.3 mm^2



3.5.3 Analysis Model

Loading condition		
Beam Diagram	SF	
	BM	

3.5.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Axial resistance	2999.75kN	2946.50 kN	1.78%
Shear resistance	1154.60kN	1134.00kN	1.78%
Bending resistance	179.28kNm	174.30 kNm	2.78%
Buckling resistance	1247.29kN	1209.00kN	3.07%

3.5.5 Detailed comparison

<table>
<tr><th>midas Gen</th><th>Example book</th></tr>
<tr><td>

1. Cross-section classification
( ). Determine classification of compression Internal Parts.
[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]
-. e = SQRT(235/fy) = 0.81
-. d/t = HTR = 3.25
-. sigma1 = 0.376 kN/mm^2.
-. sigma2 = 0.376 kN/mm^2.
-. HTR < 33*e **(Class 1 : Plastic).**

( ). Determine classification of bending and compression Internal Parts.
[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]
-. e = SQRT(235/fy) = 0.81
-. d/t = HTR = 9.50
-. sigma1 = 0.624 kN/mm^2.
-. sigma2 = -0.603 kN/mm^2.
-. Psi = [2*(Nsd/A)*(1/fy)]-1 = -0.940
-. Alpha = 0.524 > 0.5
-. HTR < 396*e/(13*Alpha-1) **(Class 1 : Plastic).**

2. Check Axial Resistance.
( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]
-. Kl/i = 64.3 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).
[Eurocode3:05 6.1, 6.2.4]
-. Nc_Rd = fy * Area / Gamma_M0 = **2999.75 kN**.

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).
N_Ed 90.00
-. ---------- = ----------- = 0.030 < 1.000 ---> O.K.
Nc_Rd 2999.75

</td><td>

1.Cross-section classification (clause 5.5.2)
$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.81$
For a RHS the compression width c may be taken as h (or b) – 3t.

Flange-internal part in compression (Table 5.2, sheet 1):
C = b - 3t = 100 – (3 X 16.0) = 52.0 mm
c/t = 52.0/16.0 = 3.25
Limit for Class 1 flange=33ε= 26.85
26.85 > 3.25 **∴flange is Class 1**

Web – internal part in compression (Table 5.2, sheet 1):
C = h - 3t = 200.0 – (3 X 16.0) = 152.0 mm
c/t = 152.0/16.0 = 9.50
Limit for Class 1 web = 33ε = 26.85
26.85 > 9.50 **∴web is Class 1**
The overall cross-section classification is therefore Class 1 (under pure compression).

2.Compression resistance of cross-section (clause 6.2.4)
The design compression resistance of the cross-section $N_{c,Rd}$
$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}}$ for class 1,2 and 3 cross-sections
$= \frac{8300 \times 355}{1.00}$ = 2946500 N = **2946.5 KN**
2946.5 KN > 90 KN ∴ acceptable

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

3. Check Bending Moment Resistance About Major Axis **( ). Calculate plastic resistance moment about major axis.** [Eurocode3:05 6.1, 6.2.5] -. Wply = 0.0005 m^3. -. Mc_Rdy = Wply * fy / Gamma_M0 = **179.28 kN-m.** **( ). Check ratio of moment resistance (M_Edy/Mc_Rdy).** M_Edy 139.20 -. ---------- = ------------- = 0.776 < 1.000 ---> O.K. Mc_Rdy 179.28	***3.Bending resistance of cross-section (clause 6.2.5)*** Maximum bending moment $M_{y,\,Ed}$ = 2.4 × 58 = 139.2 KN The design major axis bending resistance of the cross-section. $M_{c,\,y,\,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}}$ for Class 1 or 2 cross-sections $= \frac{491000 \times 355}{1.00}$ = 174.3 × 10^6 Nmm **= 174.3 KNm** 174.3 KNm > 139.2 KNm ∴ acceptable
4. Shear resistance of cross-section **( ). Calculate shear area.** [Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2] -. Avy = Area * B/(B+h) = 0.0028 m^2. -. Avz = Area * h/(B+h) = 0.0056 m^2. **( ). Calculate plastic shear resistance in local-z direction (Vpl_Rdz).** [Eurocode3:05 6.1, 6.2.6] -. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0 = 1154.60 kN **( ). Shear Buckling Check**. [Eurocode3:05 6.2.6] -. HTR < 72*e/Eta ---> **No need to check!** **( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).** (LCB = 1, POS = J) -. Applied shear force : V_Edz = 58.00 kN. V_Edz 58.00 -. ----------- = ------------ = 0.050 < 1.000 ---> **O.K.** Vpl_Rdz 1154.60	***4.Shear resistance of cross-section (clause 6.2.6)*** Maximum shear force V_{ED} = 58.0 KN The design plastic shear resistance of the cross-section $V_{p1,Rd} = \frac{A_v (f_y/3)}{\gamma_{M0}}$ Or a rolled RHS of uniform thickness, loaded parallel to the depth, the shear area A_v is given by A_v = Ah/(b + h) = 8300 × 200/(100 + 200) = 5533.3 mm^2 Shear buckling need to not be considered, provided $\frac{h_w}{t_w} \leq 72 \frac{\varepsilon}{\eta}$ for unstiffened webs η= 1.2 (from EN 1993-1-5, though the UK National Annex may specify an alternative value). h_w = (h – 2t) = 200 – (2 × 16.0) = 168 mm $72 \frac{\varepsilon}{\eta} = 72 \times \frac{0.81}{1.2} = 48.8$ Actual h_w/t_w= 200/16.0 = 12.5 12.5 ≤ 48.8 ∴ **no shear buckling check required** **1134** > 58.0 KN ∴ **shear resistance is acceptable**
5. CHECK INTERACTION OF COMBINED RESISTANCE **( ). Calculate Major reduced design resistance of benging and shear**. [Eurocode3:05 6.2.8 (6.30)] -. In case of V_Edz / Vpl_Rdz < 0.5 -. My_Rd = Mc_Rdy = 179.28 kN-m. **( ). Calculate Minor reduced design resistance of benging and shear.** [Eurocode3:05 6.2.8 (6.30)] -. In case of V_Edy / Vpl_Rdy < 0.5 -. Mz_Rd = Mc_Rdz = 105.44 kN-m. **( ). Check general interaction ratio.** [Eurocode3:05 6.2.1 (6.2)] - Class1 or Class2	***5.Cross-section resistance under Bending, Shear and axial force (clause 6.2.10)*** Provided the shear force V_{ED} is less than 50% of the design plastic shear resistance $V_{p1,Rd}$ and provided shear buckling is not a concern, than the cross-section need only satisfy the requirements for bending and axial force (clause 6.2.9). In this case $V_{ED} < 0.5\ V_{p1,Rd}$, and shear buckling is not a concern (see above). Therefore, cross-section only need be checked for bending and axial force. No reduction to the major axis plastic resistance moment

N_Ed M_Edy M_Edz -. Rmax1 = --------- + ------------ + ------------ N_Rd My_Rd Mz_Rd = 0.806 < 1.000 ---> O.K. **( ). Check interaction ratio of bending and axial force member.** [Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2 -. n = N_Ed / Npl_Rd = 0.030 -. Alpha = MIN[1.66/(1-1.13*n^2), 6.0] = 1.662 -. Beta = MIN[1.66/(1-1.13*n^2), 6.0] = 1.662 -. N_Ed < 0.25*Npl_Rd = **749.94 kN.** -. N_Ed < 0.5*hw*tw*fy/Gamma_M0 = 477.12 kN. Therefore, No allowance for the effect of axial force. -. Mny_Rd = Mply_Rd = 179275.00 kN-mm. -. Rmaxy = M_Edy / Mny_Rd = 0.776 < 1.000 ---> O.K. -. N_Ed < hw*tw*fy/Gamma_M0 = **954.24 kN.** Therefore, No allowance for the effect of axial force. -. Mnz_Rd = Mplz_Rd = 105435.00 kN-mm. -. Rmaxz = M_Edz / Mnz_Rd = 0.000 < 1.000 ---> O.K. -. Rmax2 = MAX[Rmaxy, Rmaxz] = 0.776 <1.000 ---> O.K. **( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz).** [Eurocode3:05 6.3.1.1, 6.3.1.2] -. Beta_A = Aeff / Area = 1.000 -. Lambda1 = Pi * SQRT(Es/fy) = 76.409 -. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = 1.404 -. Ncry = Pi^2*Es*Ryy / KLy^2 = **1522.48 kN.** -. Lambda_by > 0.2 and N_Ed/Ncry > 0.04 --> Need to check. -. Alphay = 0.210 -. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = **1.613** -. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = **0.416** -. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1 = **1247.29** kN. -. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A) = 0.842 -. Ncrz = Pi^2*Es*Rzz / KLz^2 = 4227.99 kN. -. Lambda_bz < 0.2 or N_Ed/Ncrz < 0.04 --> No need to check.	due to the effect of axial force is required when both of the following criteria are satisfied: $N_{ED} \le 0.25\, N_{p1,Rd}$ $N_{ED} \le \frac{0.5 h_w t_w f_y}{\gamma_{M0}}$ $0.25\, N_{p1,Rd} = 0.25 \times 2946.5 =$ **736.64 KN** 736.64 KN > 90KN ∴ equation (6.33) is satisfied $\frac{0.5 h_w t_w f_y}{\gamma_{M0}} = \frac{0.5 \times 168.0 \times (2 \times 16.0) \times 355}{\gamma_{M0}} =$ **954.2 KN** 954.2 KN > 90KN ∴ equation (6.34) is satisfied Therefore, no allowance for the effect of axial force on the major axis plastic moment resistance of the cross-section need be made. ***6.Member buckling resistance in compression (clause 6.3.1)*** $N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ for Class 1,2 or 3 cross-sections $\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}}$ but $\chi \le 1.0$ where $\Phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$ and $\lambda = \sqrt{\frac{A f_y}{N_{cr}}}$ for Class 1,2 or 3 cross-sections *Elastic critical force and non-dimensional slenderness for flexural buckling* For buckling about the major (y-y) axis, L_{cr} should be taken as the full length of the beam(AD), which is 7.2 m. For buckling about the minor (z-z)axis, L_{cr} should be taken as the maximum length between points of lateral restraint, which is 2.4 m. Thus, $N_{cr,y} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \times 210000 \times 36780000}{7200^2} = 1470 \text{ X } 10^3$ = **1470 KN** $\therefore \lambda = \sqrt{\frac{8300 \times 355}{1470 \times 10^3}} = 1.42$ $N_{cr,z} = \frac{\pi^2 EI}{L_{cr}^2} = \frac{\pi^2 \times 210000 \times 11470000}{2400^2} = 4127 \text{ X } 10^3$ = **4127 KN** $\therefore \lambda = \sqrt{\frac{8300 \times 355}{4127 \times 10^3}} = 0.84$ Selection of buckling curve and imperfection factor α For a hot-rolled RHS, use buckling curve a (Table 6.5 (Table 6.2 of EN 1993-1-1)). For curve buckling curve a, α = 0.21 (Table 6.4 (Table 6.1

of EN 1993-1-1))

Buckling curves : major (y-y)axis

$\phi_y = 0.5 \times [1 + 0.21 \times (1.42 - 0.2) + 1.42^2] =$ **1.63**

$\therefore \chi_y = \frac{1}{1.63 + \sqrt{1.63^2 - 1.42^2}} =$ **0.41**

$N_{b,y,Rd} = \frac{0.41 \times 8300 \times 355}{1.0} = 1209 \times 10^3 =$ **1209** KN

1209 KN > 90 KN

∴ major axis flexural buckling resistance is acceptable

Buckling curve: minor (z-z) axis

$\phi_z = 0.5 \times [1 + 0.21 \times (0.84 - 0.2) + 0.84^2] = 0.92$

$\therefore \chi_y = \frac{1}{0.92 + \sqrt{0.92^2 - 0.84^2}} = 0.77$

$N_{b,y,Rd} = \frac{0.77 \times 8300 \times 355}{1.0} = 2266 \times 10^3 = 2266$ KN

2266 KN > 90 KN

∴ minor axis flexural buckling resistance is acceptable

( ). Calculate equivalent uniform moment factors (Cmy,Cmz,CmLT). [Eurocode3:05 Annex A. Table A.1, A.2]

-. Cmy,0 = **0.989**
-. Cmz,0 = 1.005
-. Cmy (Default or User Defined Value) = 1.000
-. Cmz (Default or User Defined Value) = 1.000
-. CmLT (Default or User Defined Value) = 1.000

7.Member buckling resistance in combined bending and axial compression (clause 6.3.3)

Non-dimensional slenderness

From the flexural buckling check:

$\lambda_y = 1.42$ and $\lambda_z = 0.84$ ∴ $\lambda_{max} = 1.42$

From the lateral torsional buckling check:

$\lambda_{LT} = 0.23$ and $\lambda_0 = 0.23$

( ). Check interaction ratio of bending and axial compression member. [Eurocode3:05 6.3.1, 6.2.9.3 (6.61, 6.62), Annex A]

-. kyy = 1.046
-. kyz = 0.609
-. kzy = 0.681
-. kzz = 1.035
-. Xiy = 0.416
-. Xiz = 0.771
-. XiLT = 1.000
-. N_Rk = A*fy = 2999.75 kN.
-. My_Rk = Wply*fy = 179.28 kN-m.
-. Mz_Rk = Wplz*fy = 105.44 kN-m.
-. N_Ed*eNy = 0.0 (Not Slender)
-. N_Ed*eNZ = 0.0 (Not Slender)

Equivalent uniform moment factors C_{mi}

$C_{my,0} = 0.79 + 0.21\,\psi_y + 0.36(\psi_y - 0.33)\,\frac{N_{ED}}{N_{cr,y}}$

$0.79 + (0.21 \times 1.0) + 0.36 \times (1.0 - 0.33)\,\frac{90}{1470}$

= **1.01**

$C_{mz,0} = C_{mz}$ need not be considered since $M_{z,ED} = 0$.

$\varepsilon_y = \frac{M_{y,ED}}{N_{ED}} \frac{A}{W_{el,y}}$ for class 1,2 and 3 cross-sections

$= \frac{139.2 \times 10^6}{90 \times 10^3} \frac{8300}{368000} = 34.9$

$\alpha_{LT} = 1 - \frac{I_T}{I_y} \geq 1.0 = 1 - \frac{29820000}{36780000} = 0.189$

$C_{my} = C_{mz,0} + (1 - C_{my,0}) \frac{\sqrt{\varepsilon y}\,\alpha LT}{1 + \sqrt{\varepsilon y}\,\alpha LT} = 1.01 + (1 - 1.01)\,\frac{\sqrt{34.9} \times 0.189}{1 + (\sqrt{34.9} \times 0.189)}$

$= 1.01$

$C_{mLT} = C_{my}^2 \frac{\alpha LT}{\sqrt{[1-(N_{Ed}/N_{cr,z})][1-(N_{Ed}/N_{cr,T})]}}$

$= 1.01^2 \frac{0.189}{\sqrt{[1-(904127)][1-(90415502)]}}$

≥ 1.0 (but ≥ 1.0) ∴ $C_{mLT} = 1.00$

-. Rmax_LT1 = N_Ed / ----------------------------- Xiy*N_Rk/Gamma_M1
M_Edy + N_Ed*eNy

Interaction factors k_{ij}

$k_{yy} = C_{my} C_{mLT} \frac{\mu_y}{1 - N_{ED}/N_{cr,y}} \frac{1}{C_{yy}}$

$= 1.01 \times 1.00 \times \frac{0.96}{1 - 90/1470} \times \frac{1}{0.98} =$ **1.06**

```
                        +   kyy * ---------------------------------
                                    XiLT*My_Rk/Gamma_M1
                                      M_Edz + N_Ed*eNz
                        +   kyz * ----------------------------
                                      Mz_Rk/Gamma_M1

                        =  0.885  <  1.000  ---> O.K

                                    N_Ed
        -. Rmax_LT2  =   -----------------------------
                               Xiz*N_Rk/Gamma_M1
                                      M_Edy + N_Ed*eNy
                       +  kzy * ----------------------------------
                                    XiLT*My_Rk/Gamma_M1
                                    M_Edz + N_Ed*eNz
                       +  kzz * ---------------------------
                                    Mz_Rk/Gamma_M1
                       =  0.568  <  1.000  ---> O.K.

       -. Rmax   = MAX[ MAX(Rmax1, Rmax2),
                   MAX(Rmax_LT1, Rmax_LT2) ]
                 =  0.885  <  1.000  ---> O.K.
```

$k_{zy} = C_{my}C_{mLT}\frac{\mu_y}{1-N_{ED}/N_{cr,y}}\frac{1}{C_{zy}}0.6\sqrt{\frac{W_y}{W_z}}$

$= 1.01 \times 1.00 \times \frac{0.99}{1-90/1470} \times \frac{1}{0.95} \times 0.6 \times \sqrt{\frac{1.33}{1.27}}$ $=\mathbf{0.69}$

Check compliance with interaction formulae

(equations (6.61) and (6.62))

$\frac{N_{ED}}{\chi_y N_{RK}/\gamma_{M1}} + k_{yy}\frac{M_{y,ED}}{\chi_{LT} M_{y,RK}/\gamma_{M1}} + k_{zy}\frac{M_{z,Ed}}{M_{z,RK}/\gamma_{M1}} \le 1$

$\Rightarrow \frac{90}{(0.41 \times 2947)/1.0} + 1.06 \times \frac{139.2}{(0.97 \times 174.3)/1.0}$

= 0.07 + 0.87 = 0.94

0.94 ≤ 1.0 ∴ equations (6.61) is satisfied

$\frac{N_{ED}}{\chi_z N_{RK}/\gamma_{M1}} + k_{zy}\frac{M_{y,ED}}{\chi_{LT} M_{y,RK}/\gamma_{M1}} + k_{zz}\frac{M_{z,Ed}}{M_{z,RK}/\gamma_{M1}} \le 1$

$\Rightarrow \frac{90}{(0.77 \times 2947)/1.0} + 0.69 \times \frac{139.2}{(0.97 \times 174.3)/1.0}$

= 0.04 + 0.57 = 0.61

0.61 ≤ 1.0 ∴ equations (6.62) is satisfied

Therefore, a hot-rolled 200 × 100 × 16 RHS in grade S355 steel is suitable for this application.

[Reference]
L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 81-89 (Example 6.9)

3.6 Member resistance under combined bi-axial bending and axial compression

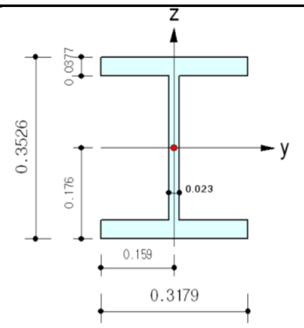
An H section member of length 4.2 m is to be designed as a ground floor column in a multi-storey building. The frame is moment resisting in-plane and pinned out-of-plane, with diagonal bracing provided in both directions. The column is subjected to major axis bending die to horizontal forces and minor axis bending due to eccentric loading from the floor beams. From the structural analysis, the design action effects of Fig.6.29 arise in the column.
Assess the suitability of a hot-rolled 305 X 305 X 240H section in grade S275 steel for this application.

3.6.1 Material Properties

Material	S275	f_y = 275N/mm^2	E_s = 210 GPa

3.6.2 Section Properties

Section Name	305 X 305 X 240H
Depth *(H)*	352.5 mm
Width *(B)*	318.4 mm
Flange Thickness *(T_f)*	37.70 mm
Web Thickness *(T_w)*	23.0 mm
Gross sectional area *(A)*	30600.0 mm^2
Shear area *(A_{sz})*	8033.0 mm^2

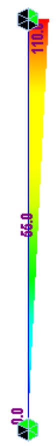


3.6.3 Analysis Model

Beam Diagram

Axial force(N_{Ed})

Bending moment ($M_{y,Ed}$)



Bending moment ($M_{z,Ed}$)

3.6.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Axial resistance	8415.00kN	8415.00kN	0.00%
Shear resistance	1168.75 kNm	1168.00kNm	0.06%
Bending resistance	1366.36kN	1275.00kN	6.69%
Reduced plastic moment resistance	770.79kNm	773.80kNm	0.39%

3.6.5 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges**. [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. b/t = BTR = 3.51 -. sigma1 = 213390.031 KPa. -. sigma2 = 141865.416 KPa. -. BTR < 9*e **(Class 1 : Plastic).** **( ). Determine classification of compression Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. d/t = HTR = 10.73 -. sigma1 = 201819.634 KPa. -. sigma2 = 23016.967 KPa. -. HTR < 33*e **(Class 1 : Plastic).**	***Cross-section classification (clause 5.5.2)*** $\varepsilon = \sqrt{235/f_y} = \sqrt{235/275} = 0.92$ Outstand flanges (Table 5.2, sheet 2): $C = (b - t_w - 2r)/2 = 132.5$ mm $c/t_f = 132.5/37.7 = 3.51$ Limit for Class 1 flange=9ε= 8.32 8.32 > 3.51 **∴flange is Class 1** Web – internal part in bending (Table 5.2, sheet 1): $C = h - 2t_f - 2r = 246.7$ mm $c/t_w = 246.7/23.0 = 10.73$ Limit for Class 1 web = 33ε = 30.51 30.51 > 10.73 **∴ web is Class 1** **The overall cross-section classification is therefore Class 1.**
2. Check Axial Resistance. **( ). Check slenderness ratio of axial compression member (Kl/i).** [Eurocode3:05 6.3.1] -. Kl/i = 51.6 < 200.0 ---> O.K. **( ). Calculate axial compressive resistance (Nc_Rd).** [Eurocode3:05 6.1, 6.2.4] -. Nc_Rd = fy * Area / Gamma_M0 = **8415.00 kN.** **( ). Check ratio of axial resistance (N_Ed/Nc_Rd).** -. N_Ed / Nc_Rd = 3440.00 / 8415.00 = 0.518 < 1.000 ---> O.K.	***Compression resistance of cross-section (clause 6.2.4)*** The design compression resistance of the cross-section $N_{c,Rd} = \frac{Af_y}{\gamma_{M0}}$ for class 1,2 and 3 cross-sections $= \frac{30600 \times 275}{1.00}$ = 8415000 N = **8415 KN** 8415 KN > 34400 KN ∴ acceptable
3. Check Bending Moment Resistance About Major Axis **( ). Calculate plastic resistance moment about major axis.** [Eurocode3:05 6.1, 6.2.5]	***Bending resistance of cross-section (clause 6.2.5)*** major (y-y) axis Maximum bending moment $M_{y,Ed}$ = 420.0 KN The design major axis bending resistance of the cross-section.

-. Wply = 0.0043 m^3.
-. Mc_Rdy = Wply * fy / Gamma_M0 = **1168.75 kN-m.**

( ). Check ratio of moment resistance (M_Edy/Mc_Rdy).

```
       M_Edy       420.00
-. ------------ = ------------- = 0.359 < 1.000 ---> O.K.
       Mc_Rdy      1168.75
```

( ). Calculate plastic resistance moment about minor axis.
[Eurocode3:05 6.1, 6.2.5]
-. Wplz = 0.0020 m^3m^3.
-. Mc_Rdz = Wplz * fy / Gamma_M0 = **536.25kN-m.**

( ). Check ratio of moment resistance (M_Edz/Mc_Rdz).

```
       M_Edz       110.00
-. ------------ = ------------ = 0.205 < 1.000 ---> O.K.
       Mc_Rdz      536.25
```

3. Shear resistance of cross-section

( ). Calculate shear area.
[Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2]
-. eta = 1.2 (Fy < 460 MPa.)
-. r = 0.0152 m.
-. Avy = Area - hw*tw = 0.0242 m^2.
-. Avz1 = eta*hw*tw = 0.0076 m^2.
-. Avz2 = Area - 2*B*tf + (tw + 2*r)*tf = 0.0086 m^2.
-. Avz = MAX[Avz1, Avz2] = 0.0086 m^2.

( ). Calculate plastic shear resistance in local-z direction (Vpl_Rdz). [Eurocode3:05 6.1, 6.2.6]
-. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0 = **1366.36kN.**

( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).
(LCB = 1, POS = J)
-. Applied shear force : V_Edz = 200.00 kN.

```
       V_Edz       200.00
-. ------------ = ------------- = 0.146 < 1.000 ---> O.K.
       Vpl_Rdz     1366.36
```

( ). Calculate plastic shear resistance in local-y direction (Vpl_Rdy). [Eurocode3:05 6.1, 6.2.6]
-. Vpl_Rdy = [Avy*fy/SQRT(3)] / Gamma_M0 = **3846.51** kN.

( ). Check ratio of shear resistance (V_Edy/Vpl_Rdy).
(LCB = 1, POS = J)
-. Applied shear force : V_Edy = 26.19 kN.

```
       V_Edy       26.19
-. ----------- = ------------ = 0.007 < 1.000 ---> O.K.
       Vpl_Rdy     3846.51
```

( ). Shear Buckling Check. [Eurocode3:05 6.2.6]
-. HTR < 72*e/Eta ---> **No need to check!**

$M_{c,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}}$ for Class 1 or 2 cross-sections

$= \frac{4247000 \times 275}{1.00} = 1168 \times 10^6$ Nmm

= **1168 KNm**

1168 KNm > 420.0 KNm ∴ acceptable

minor (z-z) axis
Maximum bending moment
$M_{y,Ed}$ = 110.0 KN
The design minor axis bending resistance of the cross-section

$M_{c,z,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{1915000 \times 275}{1.00} = 536.5 \times 10^6$ Nmm

= **536.5 KNm**

536.5 KNm > 110.0 KNm ∴ acceptable

Shear resistance of cross-section (clause 6.2.6)

The design plastic shear resistance of the cross-section

$V_{p1,Rd} = \frac{A_v (f_y/3)}{\gamma_{M0}}$

Load parallel to web
Maximum shear force
V_{ED} = 840/4.2 = 200.0 KN
For a rolled H section, loaded parallel to the web, the shear area A_v is given by

$A_v = A - 2bt_f + (t_w + r)\, t_f$ (but not less than $\eta h_w t_w$)

η= 1.2 (from Eurocode 3 –part 1.5, though the UK National Annex may specify an alternative value).

$h_w = (h - 2t_f) = 352.5 - (2 \times 37.7) = 277.1$ mm

∴ $A_v = 30600 - (2 \times 318.4 \times 37.7) + (23.0 + 15.2) \times 37.7$

$= 8033\ mm^2$

(but not less than $1.2 \times 277.1 \times 23.0 = 7648 mm^2$)

$V_{p1,Rd} = \frac{8033 \times (275/3)}{1.00} = 1275000$ N = **1275KN**

1275KN > 200 KN ∴ acceptable

Load parallel to flanges
Maximum shear force
V_{ED} = 110/3.7 = 26.2 KN
No guidance on the determination o the shear area for a rolled I or H section loaded parallel to the flanges is presented in EN 1993-1-1, though it may be assumed that adopting the recommendations provided for a welded I or H section would be acceptable.
The shear area A_v is therefore taken as

$A_w = A - \sum(h_w t_w) = 30600 - (277.1 \times 23.0)$

$= 24227\ mm^2$

$V_{p1,Rd} = \frac{24227 \times (275/3)}{1.00} = 3847000$ N = **3847KN**

3847KN > 26.2 KN ∴ acceptable

Shear buckling
Shear buckling need not be considered, provided

Note.
When calculating shear area for H sections, following equation was applied as per EN1993-1-1:2005, sub clause 6.2.6(3) a).

$A_v = A-2bt_f + (t_w+2r)t_f$

However, in the example book, following equation was applied.

$A_v = A - 2bt_f + (t_w + r)\ t_f$

For this reason, the difference in shear resistance occurred.

5. Check Interaction of Combined Resistance

( ). Check interaction ratio of bending and axial force member.

[Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2

-. n = N_Ed / Npl_Rd = 0.410
-. a = MIN[(Area-2b*tf)/Area, 0.5] = 0.214
-. Alpha = 2.000
-. Beta = MAX[5*n, 1.0] = 2.051
-. N_Ed > 0.25*Npl_Rd = 1653.88 kN.
-. N_Ed > 0.5*hw*tw*fy/Gamma_M0 = 876.64 kN.
Therefore, Allowance for the effect of axial force.
-. Mny_Rd = MIN[Mply_Rd*(1-n)/(1-0.5*a), Mply_Rd]
= **770.79 kN-m.**
-. Rmaxy = M_Edy / Mny_Rd = 0.545 < 1.000 --->O.K.
-. N_Ed > hw*tw*fy/Gamma_M0 = 2873.87 kN.
Therefore, Allowance for the effect of axial force.
-. In case of n > a
-. Mnz_Rd = Mplz_Rd * [1 - ((n-a)/(1-a))^2]
= **501.60 kN-m.**
-. Rmaxz = M_Edz / Mnz_Rd = 0.219 < 1.000 ---> O.K.

```
              [|  M_Edy  |^(Alpha)  |  M_Edz   |^(Beta) ]
-. Rmax2  =   [|-----------|         + |------------ |       ]
              [|Mny_Rd |             |  Mnz_Rd  |        ]
          = 0.341  <  1.000  --->  O.K.
```

( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz). [Eurocode3:05 6.3.1.1, 6.3.1.2]

-. Beta_A = Aeff / Area = 1.000
-. Lambda1 = Pi * SQRT(Es/fy) = 86.815
-. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = 0.234
-. Ncry = Pi^2*Es*Ryy / KLy^2 = **153942.81 kN.**
-. Lambda_by < 0.2 or N_Ed/Ncry < 0.04 --> No need to check.

-. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A) = 0.594
-. Ncrz = Pi^2*Es*Rzz / KLz^2 = **23851.54 kN.**
-. Lambda_bz > 0.2 and N_Ed/Ncrz > 0.04 --> Need to check.
-. Alphaz = 0.490

$\frac{h_w}{t_w} \leq 72\frac{\varepsilon}{\eta}$ for unstiffened webs

η= 1.2 (from Eurocode 3 –part 1.5, though the UK National Annex may specify an alternative value).

$$72\frac{\varepsilon}{\eta} = 72 \times \frac{0.92}{1.2} = 55.5$$

Actual h_w/t_w= 277.1/23.0 = 12.0

12.0 ≤ 55.5 ∴ **no shear buckling check required**

Cross-section resistance under Bending, Shear and axial force (clause 6.2.10)

Reduced plastic moment resistances (clause 6.2.9.1(5))

major (y-y)axis:

$M_{N,y,Rd} = M_{pl,y,Rd}\frac{1-n}{1-0.5a}$ (but $M_{N,y,Rd} \leq M_{pl,y,Rd}$)

Where

n = $N_{Ed}/ N_{pl,y,Rd}$ = 34400/8415 = 0.41

a = $(A - 2bt_f)/A$ = [30600 –(2 × 318.4 × 37.7)]/30600 = 0.22

$\Rightarrow M_{n,y,Rd} = 1168 \times \frac{1-0.41}{1-(0.5\times 0.22)}$ = **773.8 KNm**

773.8 KNm > 720 KNm ∴ acceptable

minor (z-z) axis

For n > a $M_{N,z,Rd} = M_{pl,y,Rd}\left[1-\left(\frac{n-a}{1-a}\right)^2\right]$

$\Rightarrow M_{N,z,Rd} = 536.5 \times \left[1-\left(\frac{0.41-0.22}{1-0.22}\right)^2\right]$ = **503.9 KNm**

503.9 KNm > 110 KNm ∴ acceptable

Cross-section check for bi-axial bending (with reduced moment resistance)

$$\left(\frac{M_{y,ED}}{M_{N,y,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{N,z,Rd}}\right)^{\beta} \leq 1$$

For I and H sections:

α=2 and β= 5n (but β ≥ 1) = (5 x 0.41) = 2.04

$$\Rightarrow\left(\frac{420}{773.8}\right)^2 + \left(\frac{110}{536.5}\right)^{2.04} = 0.33$$

0.33 ≤ 1 ∴ acceptable

Member buckling resistance in compression (clause 6.3.1)

$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ for Class 1,2 or 3 cross-sections

$\chi = \frac{1}{\phi+\sqrt{\phi^2-\lambda^2}}$ but χ ≤ 1.0

where, $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$

and $\lambda=\sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1,2 or 3 cross-sections

Elastic critical force and non-dimensional slenderness for flexural buckling

For buckling about the major (y-y) axis:

L_{cr} = 0.7L =0.7 x 4.2 = 2.94 m (see Table 6.6)

For buckling about the minor (z-z)axis:

-. Phiz = 0.5 * [1 + Alphaz*(Lambda_bz-0.2) + Lambda_bz^2]
= 0.773
-. Xiz = MIN [1 / [Phiz + SQRT(Phiz^2 - Lambda_bz^2)], 1.0]
= 0.789
-. Nb_Rdz = Xiz*Beta_A*Area*fy / Gamma_M1 = 6640.85 kN.

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd).

-. Nb_Rd = MIN[Nb_Rdy, Nb_Rdz] = **6640.85 kN.**

```
   N_Ed         3440.00
-. ---------- = ------------- = 0.518 < 1.000 ---> O.K.
   Nb_Rd        6640.85
```

L_{cr} = 1.0L = 1.0 x 4.2 = 4.20 m (see Table 6.6)

$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr}^{\ 2}} = \frac{\pi^2 \times 210000 \times 642.0 \times 10^6}{2940^2}$ = 153943 x 10^3 N

= **153943 KN**

$\therefore \lambda = \sqrt{\frac{30600 \times 275}{153943 \times 10^3}}$ = 0.23

$N_{cr,z} = \frac{\pi^2 EI_y}{L_{cr}^{\ 2}} = \frac{\pi^2 \times 210000 \times 203.1 \times 10^6}{4200^2}$ = 23863 $\times$ 10^3 N

= **23863 KN**

$\therefore \lambda = \sqrt{\frac{30600 \times 275}{23863 \times 10^3}}$ = 0.59

Buckling curves : major (y-y)axis

ϕ_y = 0.5 x [1 + 0.34 x (0.23 - 0.2) + 0.23^2] =0.53

$\therefore \chi_y = \frac{1}{0.53 + \sqrt{0.53^2 - 0.23^2}}$ = 0.99

$N_{b,y,Rd} = \frac{0.99 \times 30600 \times 275}{1.0}$ = 8314 $\times$ 10^3 =8314 KN

8314 KN > 3440 KN

∴ major axis flexural buckling resistance is acceptable

Buckling curve: minor (z-z) axis

ϕ_z = 0.5 $\times$ [1 + 0.49 $\times$ (0.59 - 0.2) + 0.59^2] = 0.77

$\therefore \chi_y = \frac{1}{0.77 + \sqrt{0.77^2 - 0.59^2}}$ = 0.79

$N_{b,y,Rd} = \frac{0.79 \times 30600 \times 275}{1.0}$ = 6640 $\times$ 10^3 = **6640 KN**

6640 KN > 3440 KN ∴ minor axis flexural buckling resistance is acceptable

[Reference]

L.Gardner and D.A.Nethercot, Designers' Guide to EN 1993-1-1, The Steel Construction Institute, Thomas Telford, 89-97 (Example 6.10)

3.7 I-section beam design under shear force and bending moment

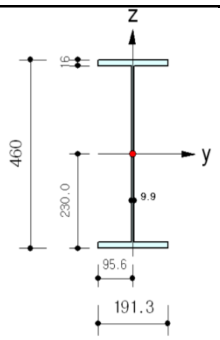
The 457 X 191 UB 82 compression member of S275 steel of Figure 3.28a is simply supported about both principle axes at each end ($L_{cr,y}$ = 12.0 m), and has a central brace which prevents lateral deflections in the minor principle plane ($L_{cr,z}$ = 6.0 m). Check the adequacy of the member for a factored axial compressive load corresponding to a nominal dead of 160 KN and a nominal imposed load of 230 KN.

3.7.1 Material Properties

Material	S275	f_y = 275 N/mm^2	E_s = 210 GPa

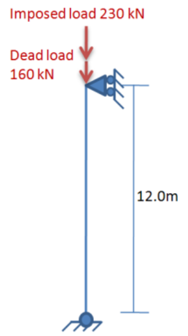
3.7.2 Section Properties

Section Name	457 X 191 UB 82
Depth *(H)*	460.0 mm
Width *(B)*	191.3 mm
Flange Thickness *(T_f)*	16 .0 mm
Web Thickness *(T_w)*	9.9 mm
Gross sectional area *(A)*	10 400.0 mm^2
Effective area *(A_{eff})*	10 067.0 mm^2



3.7.3 Analysis Model

Loading condition
&
Beam diagram



Loading condition

Axial force diagram

3.7.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Compression resistance	2768.45 kN	2765.00 kN	0.12%
Buckling resistance	845.80 kNm	844.00 kNm	0.21%

3.7.5 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. b/t = BTR = 5.03 -. sigma1 = 0.054 kN/mm^2. -. sigma2 = 0.054 kN/mm^2. -. BTR < 9*e (Class 1 : Plastic).. **( ). Determine classification of compression Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. d/t = HTR = 41.17 -. sigma1 = 0.054 kN/mm^2. -. sigma2 = 0.054 kN/mm^2. -. HTR > 42*e (**Class 4 : Slender**).	***Classifying the section.*** For S275 steel with t_f = 16 mm, f_y=275 N/mm^2 (EN 10025-2) $\varepsilon = (235/275)^{0.5} = 0.924$ $c_f/(t_f\ \varepsilon)$=[(191.3-9.9 -2 × 10.2)/2] (16.0×0.924) =5.44 < 1.4 c_w = (460.0 – 2 × 1630 – 2 × 10.2) = 407.6 mm $c_w/(t_w\ \varepsilon)$ = 407.6/(9.9 × 0.924) = 44.5 > 42 and so th web is **Class 4(slender)**
2. CALCULATE EFFECTIVE AREA. **( ). Calculate effective cross-section properties of web of Class 4 (Internal element).** [Eurocode3 Part 1-5 4.4, Table 4.1, 4.2] -. RatT = 41.1717 -. Lambda_p = RatT / [28.4*Eps*SQRT(k_sigma)] = **0.7841** -. Rho=MIN[(Lambda_p-0.055*(3+psi))/Lambda_p^2, 1.0] = **0.9175** -. sigma_max = MAX(sigma1,sigma2) = 0.054 kN/mm^2. -. sigma_min = MIN(sigma1,sigma2) = 0.054 kN/mm^2. -. r = 10.200 mm. -. Ar = 10.300 mm^2. -. dc = 407.600 mm. -. deff1 = 2*(Rho*dc) / [5 - sigma_min/sigma_max] + r = 197.187 mm. -. Aeff1 = deff1 * tw + 2*Ar = 1972.747 mm^2.	***Effective area.*** $\lambda_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{b/t}{28.4\varepsilon\sqrt{k_\sigma}} = \frac{407.6/9.9}{28.4 \times 0.924 \times \sqrt{4.0}}$ = **0.784** Ec3-1-5 4.4(2) $\rho = \frac{\lambda_p - 0.0055(3+\varphi)}{\lambda_p{}^2} = \frac{0.784 - 0.055(3+1)}{0.784^{\ 2}}$ = **0.98** Ec3-1-5 4.4(2) d- d_{eff} = (1 -0.918) × 407.6 = 33.6 mm A_{eff} = 104 × 10^2 -33.6 × 9.9 = **10067** mm^2

-. zeff1 = deff1/2 + tf = 114.593 mm.
-. deff2 = (Rho*dc) - deff1 + r = 197.187 mm.
-. Aeff2 = deff2 * tw + 2*Ar = 1972.747 mm^2.
-. zeff2 = (h+2*r) - deff2/2 + tf = 345.407 mm.

( ). Calculated effective cross-section properties of Class4 cross-section.
-. Aeff = **10067.0930** mm^2.
(for calculating axial resistance)
-. Aeffy = 10400.0000 mm^2.
-. Weffy = 1593520.8649 mm^3.
-. Aeffz = 10400.0000 mm^2.
-. Weffz = 195538.8255 mm^3.
-. eNy = 0.0000 mm.
-. eNz =2.8422e-014 mm.

2. CHECK AXIAL RESISTANCE.

( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]
-. Kl/i = 141.8 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).
[Eurocode3:05 6.1, 6.2.4]
-. Nc_Rd = fy * Aeff / Gamma_M0 = **2768.45 kN**.

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).

```
    N_Ed        561.00
-. ---------- = ------------- = 0.203 < 1.000 ---> O.K.
    Nc_Rd       2768.45
```

( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz). [Eurocode3:05 6.3.1.1, 6.3.1.2]
-. Beta_A = Aeff / Area = 0.968
-. Lambda1 = Pi * SQRT(Es/fy) = 86.815
-. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = **0.723**
-. Ncry = Pi^2*Es*Ryy / KLy^2 = 5339.87 kN.
-. Lambda_by > 0.2 and N_Ed/Ncry > 0.04
--> Need to check.
-. Alphay = 0.210
-. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = 0.817
-. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = 0.836
-. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1
= 2315.78 kN.
-. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A)
= **1.608**
-. Ncrz = Pi^2*Es*Rzz / KLz^2 = 1076.61 kN.
-. Lambda_bz > 0.2 and N_Ed/Ncrz > 0.04
--> Need to check.
-. Alphaz = 0.340
-. Phiz = 0.5 * [1 + Alphaz*(Lambda_bz-0.2) + Lambda_bz^2] = **2.031**
-. Xiz = MIN [1 / [Phiz + SQRT(Phiz^2 - Lambda_bz^2)], 1.0] = **0.306**

-. Nb_Rdz = Xiz*Beta_A*Area*fy / Gamma_M1

Cross-section compression resistance.

$$N_{c,Rd} = \frac{A_{eff}\ f_y}{\gamma_{Mo}} = \frac{10067 \times 275}{1.0} = \mathbf{2768\ KN} > 561\ KN = N_{Ed}$$

Member buckling resistance.

$$\lambda_y = \sqrt{\frac{A_{eff}\ f_y}{N_{cr,y}}} = \frac{L_{cr,y}}{i_y}\frac{\sqrt{A_{eff}\ /A}}{\lambda_1} = \frac{12000}{(18.8 \times 10)}\frac{\sqrt{10067\ /10400}}{93.9 \times 0.924} = \mathbf{0.724}$$

$$\lambda_z = \sqrt{\frac{A_{eff}\ f_y}{N_{cr,z}}} = \frac{L_{cr,z}}{i_z}\frac{\sqrt{A_{eff}\ /A}}{\lambda_1} = \frac{6000}{(4.23 \times 10)}\frac{\sqrt{10067\ /10400}}{93.9 \times 0.924} = \mathbf{1.608} > 0.724$$

Buckling will occur about the minor (z) axis. For a rolled UB section (with h/b > 1.2 and $t_f \le 40$ mm), buckling about the z-axis, use buckling curve (b) with $\alpha = 0.34$

$\Phi_z = 0.5\ [\ 1 + 0.34\ (\ 1.608 - 0.2\ ) + 1.608^2\ ] = \mathbf{2.032}$

$$\chi_z = \frac{1}{2.032 + \sqrt{2.032^2 - 1.608^2}} = \mathbf{0.305}$$

$$N_{b,z,Rd} = \frac{\chi A_{eff}\ f_y}{\gamma_{M1}} = \frac{0.305 \times 10067 \times 275}{1.0}$$

$= \mathbf{844\ KN} > 561\ KN = N_{Ed}$

And so the member is satisfactory.

= **845.80 kN.**

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd).

-. Nb_Rd = MIN[Nb_Rdy, Nb_Rdz] = 845.80 kN.

```
   N_Ed         561.00
-. ----------- = --------------- = 0.663 < 1.000 ---> O.K.
   Nb_Rd        845.80
```

[Reference]

N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 89-91 (Example 3.12.1)

3.8 Designing a UC compression member

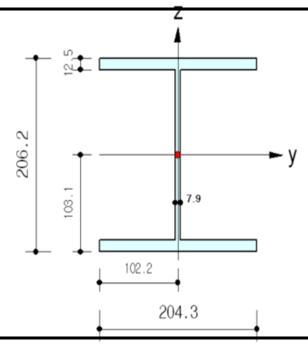
Design suitable UC of S355 steel to resist a factored axial compressive load corresponding to a nominal dead load of 160kN and a nominal imposed load of 230 kN.

3.8.1 Material Properties

Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

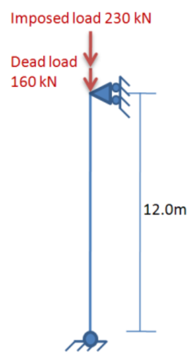
3.8.2 Section Properties

Section Name	152 x152 UC
Thickness *(T)*	206.2 mm
Width (*B*)	204.3 mm
Flange Thickness (T_f)	12.5 mm
Web Thickness (T_w)	7.9mm
Gross sectional area (*A*)	6630.0 mm^2
Shear area (A_{sz})	1876.25 mm^2



3.8.3 Analysis Model

Loading condition
&
Beam diagram



Loading condition

Axial force diagram

3.8.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Buckling resistance	615.16kN	615.00kN	0.03%

3.8.5 Detailed comparison

midas Gen

1. Cross-section classification

( ). Determine classification of compression outstand flanges.

[Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.81
-. b/t = BTR = 7.04
-. sigma1 = 0.085 kN/mm^2.
-. sigma2 = 0.085 kN/mm^2.
-. BTR < 9*e (Class 1 : Plastic).

( ). Determine classification of compression Internal Parts.

[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.81
-. d/t = HTR = 20.35
-. sigma1 = 0.085 kN/mm^2.
-. sigma2 = 0.085 kN/mm^2.
-. HTR < 33*e (Class 1 : Plastic).

2. CHECK AXIAL RESISTANCE.

( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]

-. Kl/i = 134.7 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).
[Eurocode3:05 6.1, 6.2.4]

-. Nc_Rd = fy * Area / Gamma_M0 = 2353.65 kN.

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).

```
     N_Ed         561.00
-. ------------ = -------------- = 0.238 < 1.000 ---> O.K.
     Nc_Rd        2353.65
```

( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz). [Eurocode3:05 6.3.1.1, 6.3.1.2]

-. Beta_A = Aeff / Area = 1.000
-. Lambda1 = Pi * SQRT(Es/fy) = 76.409
-. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A)
= **1.763**
-. Ncry = Pi^2*Es*Ryy / KLy^2 = 757.08 kN.
-. Lambda_by > 0.2 and N_Ed/Ncry > 0.04
--> Need to check.
-. Alphay = **0.340**
-. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = **2.319**
-. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = **0.261**
-. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1
= **615.16 kN.**

Example book

Target area and first section choice.

Assume $f_y = 355$ N/mm^2 and $\chi = 0.5$

$A \geq 561 \times 10^3 /(0.5 \times 355) = 3161$ mm^2

Try a 152 × 152 UC 30 with A = 38.3 cm^2,

$i_y = 6.76$ cm, $i_z = 3.83$ cm, $t_f = 9.4$ mm.

$\varepsilon = (235/355)^{0.5} = 0.814$

$$\lambda_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \frac{L_{cr,y}}{i_y}\frac{1}{\lambda_1} = \frac{12000}{(6.76 \times 10)}\frac{1}{93.9 \times 0.814} = 2.322$$

$$\lambda_z = \sqrt{\frac{Af_y}{N_{cr,z}}} = \frac{L_{cr,z}}{i_z}\frac{1}{\lambda_1} = \frac{6000}{(3.83 \times 10)}\frac{1}{93.9 \times 0.814} = 2.050 < 2.322$$

Buckling will occur about the major (y) axis. For a rolled UC section (with h/b ≤ 1.2 and $t_f \leq 100$ mm), buckling about the y-axis, use buckling curve (b) with α = 0.34

$\Phi_y = 0.5\,[\,1 + 0.34\,(\,2.322 - 0.2\,) + 2.322^2\,] = 3.558$

$$\chi_y = \frac{1}{3.558 + \sqrt{3.558^2 - 2.322^2}} = 0.160$$

which is much less than the guessed value of 0.5.

Second section choice.

Guess $\chi = (0.5 + 0.160)/2 = 0.33$

$A \geq 561 \times 10^3 /(0.33 \times 355) = 4789$ mm^2

Try a 203 × 203 UC 52 with A = 66.3 cm^2, $i_y = 8.91$ cm, , t_f = 12.5 mm.

For S255 steel with $t_f = 12.5$ mm, $f_y = 355$ N/mm^2

$\varepsilon = (235/355)^{0.5} = 0.814$

$c_f/(t_f\,\varepsilon) = [(204.3 - 7.9 - 2 \times 10.2) / 2]\,(12.5 \times 0.814)$
$= 8.65 < 14$

$c_w/(t_w\,\varepsilon) = (206.2 - 2 \times 12.5 - 2 \times 10.2)$
$/ (7.9 \times 0.814) = 25.0 > 42$

and so the cross-section is fully effective.

$$\lambda_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \frac{L_{cr,y}}{i_y}\frac{1}{\lambda_1} = \frac{12000}{(8.91 \times 10)}\frac{1}{93.9 \times 0.814} = \mathbf{1.763}$$

For a rolled UC section (with h/b > 1.2 and $t_f \leq 100$ mm), buckling about the y-axis, use buckling curve (b) with α = **0.34**

$\Phi_y = 0.5\,[\,1 + 0.34\,(\,1.763 - 0.2\,) + 1.763^2\,] = \mathbf{2.320}$

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd). -. Nb_Rd = MIN[Nb_Rdy, Nb_Rdz] = 615.16 kN. N_Ed 561.00 -. ----------- = --------------- = 0.912 < 1.000 ---> O.K. Nb_Rd 615.16	$\chi_y = \frac{1}{2.032+\sqrt{2.032^2-1.763^2}} = \mathbf{0.261}$ $N_{b,y,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0.261 \times 66.3 \times 10^2 \times 355}{1.0}$ = **615 KN** > 561 KN = N_{Ed} and so the 203 × 203 UC 52 is satisfactory. [Reference] N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 91-92 (Example 3.12.2)

3.9 Design an RHS compression

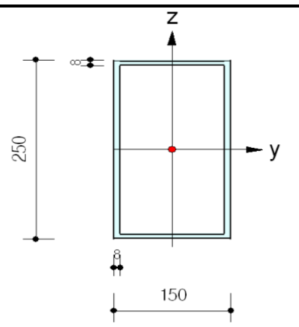
Design a suitable hot – finished RHS of S355 steel to resist a factored axial compressive load corresponding to a nominal dead load of 160kN and a nominal imposed load of 230 kN.

3.9.1 Material Properties

Material	S275	f_y = 275 N/mm^2	E_s = 210 GPa

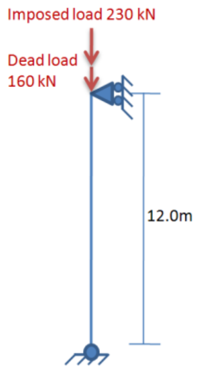
3.9.2 Section Properties

Section Name	250 X 150 X 8 RHS
Depth *(H)*	250.0 mm
Width *(B)*	150.0 mm
Flange Thickness *(T_f)*	8.0 mm
Web Thickness *(T_w)*	8.0 mm
Gross sectional area *(A)*	6080.0 mm^2
i_y	91.7mm



3.9.3 Analysis Model

Loading condition
&
Beam diagram



Loading condition

Axial force diagram

3.9.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Buckling resistance	610.0kN	640.0kN	4.92%

3.9.5 Detailed comparison

midas Gen

1. Cross-section classification

( ). Determine classification of compression Internal Parts.

[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.81
-. d/t = HTR = 28.25
-. sigma1 = 0.095 kN/mm^2.
-. sigma2 = 0.095 kN/mm^2.
-. HTR < 38*e (Class 2 : Compact).

2. Check axial resistance.

( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]

-. Kl/i = 132.2 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).

[Eurocode3:05 6.1, 6.2.4]

-. Nc_Rd = fy * Area / Gamma_M0 = 2101.60 kN.

( ). Check ratio of axial resistance (N_Ed/Nc_Rd).

```
       N_Ed           561.00
-. ------------ = -------------- = 0.267 < 1.000 ---> O.K.
       Nc_Rd          2101.60
```

( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz). [Eurocode3:05 6.3.1.1, 6.3.1.2]

-. Beta_A = Aeff / Area = 1.000
-. Lambda1 = Pi * SQRT(Es/fy) = 76.409
-. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = **1.730**
-. Ncry = Pi^2*Es*Ryy / KLy^2 = 703.83 kN.
-. Lambda_by > 0.2 and N_Ed/Ncry > 0.04 --> Need to check.
-. Alphay = **0.210**
-. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = 2.156
-. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = **0.290**
-. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1 = **610.18** kN.
-. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A) = **1.283**

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd).

-. Nb_Rd = MIN[Nb_Rdy, Nb_Rdz] = 610.18 kN.

```
      N_Ed        561.00
-. ----------- = --------------- = 0.919 < 1.000 ---> O.K.
      Nb_Rd       610.18
```

Example book

Guess $\chi = 0.3$

$$A \geq 561 \times 10^3/(0.3 \times 355) = 5268\,\mathrm{mm}^2$$

Try a $250 \times 150 \times 8$ RHS, with $\mathrm{A} = 60.8\mathrm{cm}^2,\ i_y = 9.17cm, i_z = 6.15\ cm, t = 8.0mm.$

For S355 steel with $t = 8mm,\ \mathrm{f_y} = 355\,N/\mathrm{mm}^2$

EN 10025-2

$$\varepsilon = (235/355)^{0.5} = 0.814$$

$$c_w/(t\varepsilon) = \frac{(250.0 - 2 \times 8.0 - 2 \times 4.0)}{(8.0 \times 0.814)}$$

= 34.7 < 42 T5.2

And so the cross-section is fully effective.

$$\bar{\lambda}_y = \sqrt{\frac{A\mathrm{f_y}}{\mathrm{N_{cr,y}}}} = \frac{\mathrm{L_{cr,y}}}{\mathrm{i_y}}\frac{1}{\lambda_1} = \frac{12000}{(9.18 \times 10)}\frac{1}{93.9 \times 0.814}$$

= 1.710

$$\bar{\lambda}_z = \sqrt{\frac{A\mathrm{f_y}}{\mathrm{N_{cr,z}}}} = \frac{\mathrm{L_{cr,z}}}{\mathrm{i_z}}\frac{1}{\lambda_1} = \frac{6000}{(6.15 \times 10)}\frac{1}{93.9 \times 0.814}$$

= 1.276

Buckling will occur about the major (ν) axis. For a hot-
Finished RHS, use bucking curve (a) with α=**0.21**

$$\emptyset_y = 0.5[1 + 0.21(0.710 - 0.2) + 1.710^2]$$
$$= \mathbf{2.121} \geq 156$$

$$\chi_y = \frac{1}{2.121 + \sqrt{2.121^2 - 1.710^2}} = \mathbf{0.296}$$

$$N_{b,y,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0.296 \times 60.8 \times 10^3 \times 355}{1.0}$$
$$= \mathbf{640}kN > 561kN = \mathrm{N_{Ed}}$$

and so the $250 \times 150 > 8$ RHS is satisractory

Note.
The difference in buckling resistance occurred since currently midas Gen does not consider 'r' value for rolled box section. This can be improved in the future version.

[Reference]
N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 92-93 (Example 3.12.3)

3.10 Compression resistance of a Class 4 compression member

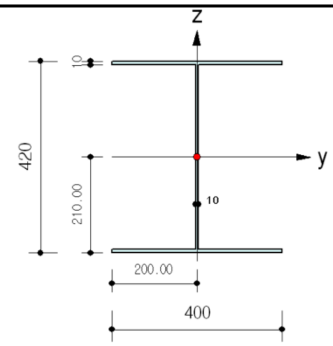
Determine the compression resistance of the cross-section of the member shown in Figure the figure below.

3.10.1 Material Properties

Material	S275	f_y = 275 N/mm^2	E_s = 210 GPa

3.10.2 Section Properties

Section Name	420 X 400
Depth *(H)*	420.0 mm
Width *(B)*	400.0 mm
Flange Thickness *(T_f)*	10.0 mm
Web Thickness *(T_w)*	10.0 mm
Gross sectional area *(A)*	12054.9 mm^2
Effective area (A_{eff})	9216.0mm^2



3.10.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Axial resistance	3271.45kN	3272.00kN	0.02%

3.10.4 Detailed comparison

midas Gen

1. Cross-section classification

( ). Determine classification of compression outstand flanges.[Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.81
-. b/t = BTR = 18.70
-. sigma1 = 0.047 kN/mm^2.
-. sigma2 = 0.047 kN/mm^2.
-. BTR > 14*e (**Class 4 : Slender**).

( ). Determine classification of compression Internal Parts.
[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]

-. e = SQRT(235/fy) = 0.81
-. d/t = HTR = 38.40
-. sigma1 = 0.047 kN/mm^2.
-. sigma2 = 0.047 kN/mm^2.
-. HTR > 42*e (**Class 4 : Slender**).

2. Calculate Effective Area

( ). Calculate effective cross-section properties of left-top

Example book

Classifying the section plate elements.

t_f = 10 mm, t_w = 10 mm, f_y = 355 N/mm^2
En 10025-2

$\varepsilon = (235/355)^{0.5} = 0.814$

$c_f/(t_f\,\varepsilon) = (400/2 - 10/2 - 8)\,(10 \times 0.814)$
$= 23.0 > 14$ and so the flange is Class 4.

$c_w/(t_w\,\varepsilon) = (420 - 2 \times 10 - 2 \times 8) / (10 \times 0.814)$
$= 47.2 > 42$ and so the web is **Class 4.**

Effective flange area.

$k_{\sigma f} = 0.43$

$\lambda_{pf} = \dfrac{(\frac{400}{2}-\frac{10}{2}-8)/10}{28.4 \times 0.814 \times \sqrt{0.43}} = \mathbf{1.23} > 0.748$

$\rho_f = (1.23 - 0.188)/1.23^2 = \mathbf{0.687}$

$A_{eff\,f} = 0.687 \times 4 \times (400/2 - 10/2 - 8) \times 10$
$+ (10 + 2 \times 8) \times 10 \times 2$
$= \mathbf{5658}$ mm^2

flange of Class 4 (Outstand element).

[Eurocode3 Part 1-5 4.4, Table 4.1, 4.2]

-. RatT = 18.7000
-. Lambda_p = RatT / [28.4*Eps*SQRT(k_sigma)] = **1.2342**
-. Rho = MIN[(Lambda_p-0.188) / Lambda_p^2, 1.0] = **0.6868**
-. sigma_max = MAX(sigma1, sigma2) = 0.047 kN/mm^2.
-. sigma_min = MIN(sigma1, sigma2) = 0.047 kN/mm^2.
-. r = 13.000 mm.
-. bc = 187.000 mm.
-. beff = Rho*bc + r = 141.439 mm.
-. Aeff = beff * tf = **1414.395** mm^2.
-. yeff = beff/2 = 70.720 mm.

Effective flange area

-. Aeff * 4 = **5657.58** mm^2

( ). Calculate effective cross-section properties of web of Class 4 (Internal element).

[Eurocode3 Part 1-5 4.4, Table 4.1, 4.2]

-. RatT = 38.4000
-. Lambda_p = RatT / [28.4*Eps*SQRT(k_sigma)] = **0.8309**
-. Rho = MIN[(Lambda_p-0.055*(3+psi)) / Lambda_p^2, 1.0] = **0.8848**
-. sigma_max = MAX(sigma1, sigma2) = 0.047 kN/mm^2.
-. sigma_min = MIN(sigma1, sigma2) = 0.047 kN/mm^2.
-. r = 8.000 mm.
-. Ar = 0.000 mm^2.
-. dc = 384.000 mm.
-. deff1 = 2*(Rho*dc) / [5 - sigma_min/sigma_max] + r = 177.889 mm.
-. Aeff1 = deff1 * tw + 2*Ar = 1778.888 mm^2.
-. zeff1 = deff1/2 + tf = 98.944 mm.
-. deff2 = (Rho*dc) - deff1 + r = 177.889 mm.
-. Aeff2 = deff2 * tw + 2*Ar = **1778.888** mm^2.
-. zeff2 = (h+2*r) - deff2/2 + tf = 321.056 mm.

( ). Calculated effective cross-section properties of Class4 cross-section.

-. Aeff = **9215.3548** mm^2. (for calculating axial resistance)

3. Check Axial Resistance

( ). Check slenderness ratio of axial compression member (Kl/i). [Eurocode3:05 6.3.1]

-. Kl/i = 53.1 < 200.0 ---> O.K.

( ). Calculate axial compressive resistance (Nc_Rd).

[Eurocode3:05 6.1, 6.2.4]

-. Nc_Rd = fy * Aeff / Gamma_M0 = **3271.45** kN.

Effective web area.

$k_{\sigma,w}$ = 4.0

$\lambda_{p,w} = \frac{(420-2\times10-2\times8)/10}{28.4\times0.814\times\sqrt{4.0}}$ = **0.831** > 0.673

ρ_w = {0.831 – 0.055 × (3 +!1)} / 0.831^2 =**0.885**

$A_{eff,w}$ = 0.885 × (420 - 2 ×10 -2 × 8) × (10 + 8 × 10 × 2)

= 3558 mm^2

compression resistance.

A_{eff} = 5658 + 3558 = **9216** mm^2

$N_{c,Rd}$ = 9216 × 355/1.0 N =**3272** KN.

[Reference]

N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 141-142 (Example 4.9.1)

3.11 Section moment resistance of a Class 3 I-beam

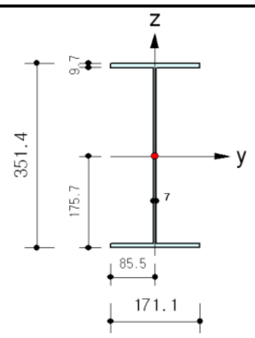
Determine the section moment resistance and examine the suitability for plastic design of the 356 X 171 UB 45 of S355 steel shown in the figure below.

3.11.1 Material Properties

Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

3.11.2 Section Properties

Section Name	356 X 171 UB 45
Depth *(H)*	351.4 mm
Width *(B)*	171.1 mm
Flange Thickness *(T_f)*	9.7 mm
Web Thickness *(T_w)*	7.0 mm
Gross sectional area *(A)*	573,000 mm^2
Plastic section modulus (W_{pl})	775.0 cm^3



3.11.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Moment resistance	275.12kNm	275.10kNm	0.01%

3.11.4 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. b/t = BTR = 7.41 -. sigma1 = 358215.065 KPa. -. sigma2 = 358215.065 KPa. -. BTR < 10*e (**Class 2 : Compact**). **( ). Determine classification of bending Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. d/t = HTR = 44.51	***Classifying the section- plate elements***. t_f = 9.7 mm, t_w = 7.0 mm, f_y = 355 N/mm^2 En 10025-2 $\varepsilon = \sqrt{(235/355)^{0.5}} = 0.814$ $c_f/(t_f\ \varepsilon)$ = (171.1/2 – 7.0/2 –10.2) (9.7 × 0.814) = 9.1 > 9 and so the flange is **Class 2**. $c_w/(t_w\ \varepsilon)$ = (351.4 – 2 × 9.7 - 2 × 10.2) / (7.0 × 0.814) = 54.7 > 72 and so the web is **Class 1**.

-. sigma1 = 652365.952 KPa. -. sigma2 = -652365.952 KPa. -. HTR < 72*e (**Class 1 : Plastic**).	
2. Check Bending Moment Resistance About Major Axis. **( ). Calculate plastic resistance moment about major axis.** [Eurocode3:05 6.1, 6.2.5] -. Wply = 0.0008 m^3. -. Mc_Rdy = Wply * fy / Gamma_M0 = **275.12** kN-m.	**section moment resistance** The cross-section is Class 2 and therefore unsuitable for plastic design. $M_{c,Rd}$ $775 \times 10^3 \times 355/1.0$ N = **275.1** KNm. [Reference] N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 142-143 (Example 4.9.2)

3.12 Section moment resistance of a Class 4 box beam

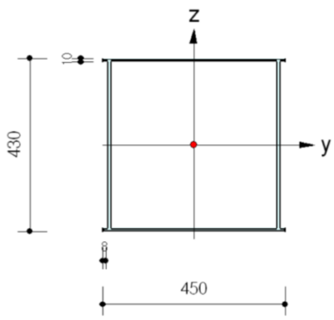
Determine the section moment resistance of the welded-box section beam of S355 steel shown in the figure below. The weld size is 6 mm.

3.12.1 Material Properties

Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

3.12.2 Section Properties

Section Name	RHS 430 X 450
Depth *(H)*	430.0 mm
Width *(B)*	450.0 mm
Flange Thickness *(T_f)*	10.0 mm
Web Thickness *(T_w)*	8.0 mm
Gross sectional area *(A)*	15560.0 mm^2
Shear area (A_{sz})	6880.00 mm^2



3.12.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Moment resistance	729.99kN	729.90kNm	0.01%

3.12.4 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. d/t = HTR = 41.00 -. sigma1 = 0.108 kN/mm^2. -. sigma2 = 0.108 kN/mm^2. -. HTR > 42*e (**Class 4 : Slender**). **( ). Determine classification of bending Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. d/t = HTR = 51.75 -. sigma1 = 0.099 kN/mm^2. -. sigma2 = -0.099 kN/mm^2. -. HTR < 72*e (**Class 1 : Plastic**).	***Classifying the section- plate elements***. t_f = 10 mm, t_w = 8 mm, f_y = 355 N/mm^2 En 10025-2 $\varepsilon = \sqrt{(235/355)}^{0.5} = 0.814$ $c_f/(t_f\ \varepsilon)$ = 410 (10 X 0.814) = 50.4 > 42 and so the flange is **Class 4.** $c_w/(t_w\ \varepsilon)$ = (430 – 2 X 10- 2 X 6) / (8 X 0.814) = 61.1 > 72 and so the web is **Class 1.** The cross-section is therefore Class 4 since the flange is **Class 4**.

2. Calculate Effective Section Modulus About Major Axis.

( ). Calculate buckling factor of internal compression element.

[Eurocode3 Part 1-5 4.4, Table 4.1]

-. In case of Psi = 1.0
-. k_sigma = 4.0000

( ). Calculate effective cross-section properties of top flange of Class 4 (Internal element).

[Eurocode3 Part 1-5 4.4, Table 4.1, 4.2]

-. RatT = 41.0000
-. Lambda_p = RatT / [28.4*Eps*SQRT(k_sigma)] = 0.8872
-. Rho = MIN[(Lambda_p-0.055*(3+psi)) / Lambda_p^2, 1.0] = 0.8477
-. sigma_max = MAX(sigma1, sigma2) = 0.108 kN/mm^2.
-. sigma_min = MIN(sigma1, sigma2) = 0.108 kN/mm^2.
-. r = 40.000 mm.
-. bc = 410.000 mm.
-. beff = Rho*bc + r = 387.537 mm.
-. Aeff = beff * tf = **3875.367 mm^2.**
-. yeff = beff/2 = 193.768 mm.

( ). Calculate cross-section properties of bottom flange.

-. r = 40.000 mm.
-. bc = 410.000 mm.
-. beff = bc + r = 450.000 mm.
-. Aeff = beff * tf = **4500.000 mm^2**.
-. yeff = beff/2 = 225.000 mm.

( ). Calculate cross-section properties of left web.

-. r = 0.000 mm.
-. Ar = 0.000 mm^2.
-. dc = 410.000 mm.
-. deff = dc + r = 410.000 mm.
-. Aeff = deff * tw + 4*Ar = **3280.000 mm^2**.
-. zeff = (h+2*r) - deff/2 = 215.000 mm.

( ). Calculate cross-section properties of right web.

-. r = 0.000 mm.
-. Ar = 0.000 mm^2.
-. dc = 410.000 mm.
-. deff = dc + r = 410.000 mm.
-. Aeff = deff * tw + 4*Ar = **3280.000 mm^2.**
-. zeff = (h+2*r) - deff/2 = 215.000 mm.

( ). Calculated effective cross-section properties of Class4 cross-section.

-. Aeffy = **14935.3672** mm^2.
-. Weffy = **2056307.7227** mm^3.

Effective cross-section

$k_\sigma = 4.0$

$\lambda_p = \frac{410}{10 \times 28.4 \times 0.814 \times \sqrt{4.0}} = 0.887 > 0.673$

$\psi = 1$

$\rho = (0.887 - 0.055\ 8 \times (3+1)) / 0.887^2 = 0.848$

$b_{c,eff} = 0.848 \times 410 = 347.5$ mm

$A_{eff} = (450 - 410 + 347.5) \times 10 + (450 \times 10) + 2 \times (430 - 2 \times 10) \times 8 = \mathbf{14935}$ mm^2

$14935 \times z_c = (450 - 410 + 347.5) \times 10 \times (430 - 10/2) + 450 \times 10 \times 10/2 + 2 \times (430 - 2 \times 10) \times 8 \times 430/2$

$z_c = \mathbf{206.2}$ mm

$I_{eff} = (450 - 410 + 347.5) \times 10 \times (430 - 10/2 - 206.2)^2 + 450 \times 10 \times (206.2 - 10/2)^2 + 2 \times (430 - 2 \times 10)^3 \times 8/12 + 2 \times (430 - 2 \times 10) \times 8 \times (430/2 - 206.2)^2$ mm^4

$= \mathbf{460.1 \times 10^6\ mm^4}$

4**. *CHECK BENDING MOMENT RESISTANCE ABOUT MAJOR AXIS.***	***Section moment resistance***
( ). Calculate local buckling resistance moment about major axis. [Eurocode3:05 6.1, 6.2.5] -. Weffy = 2056307.7227 mm^3. -. Mc_Rdy = Weffy * fy / Gamma_M1 = **729989.24 kN-mm.**	$W_{eff,\,min}$ = 460.1 $\times$ 10^6 / (430 – 206.2) = 2.056 X 10^6 mm^3 $M_{c,Rd}$ = 2.056 X 10^6 $\times$ 355/1.0 N = **729.9** KNm. [Reference] N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 143-144 (Example 4.9.3)

3.13 Section moment resistance of a slender plate girder

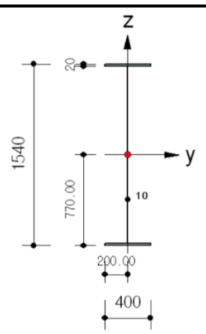
Determine the section moment resistance of the welded plate girder of S355 steel shown in the figure below. The weld size is 6 mm.

3.13.1 Material Properties

Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

3.13.2 Section Properties

Section Name	1540x400
Depth *(H)*	1540.0 mm
Width *(B)*	400.0 mm
Flange Thickness *(T_f)*	20.0 mm
Web Thickness *(T_w)*	10.0 mm
Gross sectional area *(A)*	31030.9 mm^2
Effective area (A_{eff})	14.48 x 10^6 mm^2



3.13.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Moment resistance	4996.42kNm	4996.00kNm	0.01%

3.13.4 Detailed comparison

midas Gen	Example book
1. Cross-section classification	t_f = 20 mm, t_w = 10 mm, f_y = 345 N/mm^2
	$\varepsilon = \sqrt{(235/345)}^{0.5} = 0.825$
( ). Determine classification of compression outstand flanges.	$c_f/(t_f\ \varepsilon)$ = (400/2 – 10/2 – 6) (20×0.825)
[Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5]	= 11.5 > 10 but < 14, and so the flange is Class 3.
-. e = SQRT(235/fy) = 0.83	$c_w/(t_w\ \varepsilon)$ = (1540 – 2× 20 – 2×6) / (10×0.825)
-. b/t = BTR = 9.45	= 180.3 > 124 and so the web is Class 4.
-. sigma1 = 0.016 kN/mm^2.	
-. sigma2 = 0.016 kN/mm^2.	A conservative approximation for the cross-section moment resistance may be obtained by ignoring the web completely,
-. BTR < 14*e (Class 3 : Semi-compact).	so that
	$M_{c,Rd} = M_f$ = (400×20)× (1540 – 20)×345/1.0 N
( ). Determine classification of bending Internal Parts.	= **4195** KNm.
[Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5]	
-. e = SQRT(235/fy) = 0.83	A higher resistance may be calculated by determining the
-. d/t = HTR = 148.80	
-. sigma1 = 0.015 kN/mm^2.	
-. sigma2 = -0.015 kN/mm^2.	
-. HTR > 124*e (Class 4 : Slender)	

2. Calculate Effective Section Modulus About Major Axis.

( ). Calculate cross-section properties of left-top flange.

[Eurocode3 Part 1-5 4.4, Table 4.1, 4.2]

- -. r = 11.000 mm.
- -. bc = 189.000 mm.
- -. beff = bc + r = 200.000 mm.
- -. Aeff = beff * tf = 4000.000 mm^2.
- -. yeff = beff/2 = 100.000 mm.

( ). Calculate cross-section properties of right-top flange.

- -. beff = bc + r = 200.000 mm.
- -. Aeff = beff * tf = 4000.000 mm^2.
- -. yeff = beff/2 = 100.000 mm.

( ). Calculate cross-section properties of left-bottom flange.

- -. beff = bc + r = 200.000 mm.
- -. Aeff = beff * tf = 4000.000 mm^2.
- -. yeff = beff/2 = 100.000 mm.

( ). Calculate cross-section properties of right-bottom flange.

- -. beff = bc + r = 200.000 mm.
- -. Aeff = beff * tf = 4000.000 mm^2.
- -. yeff = beff/2 = 100.000 mm.

( ). Calculate buckling factor of internal compression element.

- -. In case of Psi = -1.0
- -. k_sigma = 23.9000

( ). Calculate effective cross-section properties of web

- -. RatT =148.8000
- -. Lambda_p = RatT / [28.4*Eps*SQRT(k_sigma)] = 1.2986
- -. Rho = MIN[(Lambda_p-0.055*(3+psi)) / Lambda_p^2, 1.0] = 0.7049
- -. sigma_max = MAX(sigma1, sigma2) = 0.015 kN/mm^2.
- -. sigma_min = MIN(sigma1, sigma2) = -0.015 kN/mm^2.
- -. r = 6.000 mm.
- -. Ar = 0.000 mm^2.
- -. dc=(h*sigma_max) / (sigma_max-sigma_min) =744.0mm.
- -. deff1 = 0.4*Rho*dc + r = 215.764 mm.

effective width of the web.

$\psi = 1$

$k_\sigma = 23.9$

$\lambda_p = \frac{(1540 - 2\times 20 - 2\times 6)/10}{28.4 \times 0.825 \times \sqrt{23.9}} = 1.299$

$\rho = (1.299 - 0.055 \times (3-1)) / 1.299^2 = 0.705$

$b_c = (1540 - 2\times 20 - 2\times 6) / \{1 - (-1)\} = 744.0$ mm

$b_{eff} = 0.705\times 744.0 = 524.4$ mm

$b_{e1} = 0.4\times 524.4 = 209.8$ mm

$b_{e2} = 0.6\times 524.4 = 314.6$ mm

and the ineffective width of the web is

$b_c - b_{e1} - b_{e2} = 744.0 - 209.8 - 314.6 = 219.6$ mm

$A_{eff} = (1540 - 2\times 20 - 219.6)\times 10 + 2\times 400 \times 20$
$= \mathbf{28804}$ mm^2

$28804 \times z_c = (2\times 400\times 20 + (1540 - 2\times 20)\times 10) \times (1540/2) - 219.6 \times 10 \times (1540 - 20 - 6 - 209.8 - 219.6/2)$

$z_c = 737.6$ mm

$I_{eff} = (400\times 20)\times(1540 - 10 - 737.6)^2 + (400\times 20)\times(737.6 - 10)^2 + (1540 - 2\times 20)^3\times 10/12 + (1540 - 2\times 20)\times 10\times(1540/2 - 737.6)^2 - 219.6^3\times 10/12 - 219.6\times 10\times(1540 - 20 - 6 - 209.8 - 219.6/2 - 737/6)^2$ mm^4

$= 11.62 \times 10^9$ mm^4

$W_{eff} = 11.62\times 10^9 / (1540 - 737.6) = \mathbf{14.48\times 10^6}$ mm^3

$M_{c,Rd} = 14.48\times 10^6 \times 345/1.0$ N = **4996** KNm.

-. Aeff1 = deff1 * tw + 2*Ar = **2157.639** mm^2.
-. zeff1 = (h+2*r) - deff1/2 + tf = 1412.118 mm.
-. deff2 = 0.6*Rho*dc + (h-dc) + r = 1064.646 mm.
-. Aeff2 = deff2 * tw + 2*Ar = **10646.458** mm^2.
-. zeff2 = deff2/2 + tf = 552.323 mm.

( ). Calculated effective cross-section properties of Class4 cross-section.
-. Aeffy = **28804.0968** mm^2.
-. Weffy = **14482394.1334** mm^3.

4. Check Bending Moment Resistance About Major Axis

( ). Calculate local buckling resistance moment about major axis. [Eurocode3:05 6.1, 6.2.5]
-. Weffy = 14482394.1334 mm^3.
-. Mc_Rdy = Weffy * fy / Gamma_M1
= **4996425.98** kN-mm**.**

[Reference]

N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 144-145 (Example 4.9.4)

3.14 Shear buckling resistance of an unstiffened plate girder web

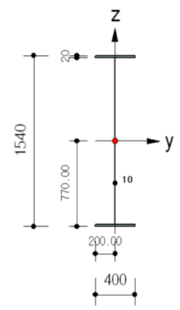
Determine the shear buckling resistance of the unstiffened plate girder web of S355 steel shown in the figure below

3.14.1 Material Properties

Material	S355	f_y = 355 N/mm^2	E_s = 210 GPa

3.14.2 Section Properties

Section Name	1540x400
Depth *(H)*	1540.0 mm
Width *(B)*	400.0 mm
Flange Thickness *(T_f)*	20.0 mm
Web Thickness *(T_w)*	10.0 mm
Gross sectional area *(A)*	31030.9 mm^2
Effective area (A_{eff})	14.48 x 10^6 mm^2



3.14.3 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	1195.86kN	1196.00kN	0.01%

3.14.4 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. b/t = BTR = 9.45 -. sigma1 = 0.016 kN/mm^2. -. sigma2 = 0.016 kN/mm^2. -. BTR < 14*e (**Class 3 : Semi-compact**). **( ). Determine classification of bending Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.81 -. d/t = HTR = 148.80 -. sigma1 = 0.015 kN/mm^2. -. sigma2 = -0.015 kN/mm^2.	t_f = 20 mm, t_w = 10 mm, f_y = 345 N/mm^2 $\varepsilon = \sqrt{(235/355)}^{0.5} = 0.814$ η =1.2 h_w = 1540 – 2 × 20 = 1500 mm $\eta h_w /(t_w\, \varepsilon)$ = 1.2× 1500 / (10 ×0.814) = 221.2 > 72 and so the web is slender $\alpha/ h_w = \infty / h_w = \infty$, $k_{\tau st} = 0$ k_τ = 5.34 τ_{cr} = 5.34 X 190000 X (10/1500)^2 = 45.1 N/mm^2 λ_w = 0.76 X $\sqrt{(355/45.1)}$ = **2.132** > 1.08 Assuming that there is a non-rigid end post, then χ_w = 0.83 /2.132 = **0.389**

-. HTR > 124*e (**Class 4 : Slender**) ***2. Check Shear Resistance.*** **( ). Calculate shear buckling resistance in local-z direction (Vbl_Rdz).** [Eurocode3:05 6.1, 6.2.6, EN 1993-1-5:2004 5.2] -. Eta = 1.20 -. Lambda_w = hw / (86.4*tw*e) = **2.1338** -. Chi_w = 0.83/Lambda_w = **0.39** -. Vbw_Rdz = Chi_w*fy*hw*tw / [sqrt(3)*Gamma_M1] = **1195.86** kN.	Neglecting any contribution from the flanges, $V_{b,Rd} = V_{bw,Rd} = \frac{(0.389 \times 355 \times 1500 \times 10)}{\sqrt{3} \times 1.0}$ N = **1196** KN [Reference] N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 145-146 (Example 4.9.5)

3.15 Checking a simply supported beam

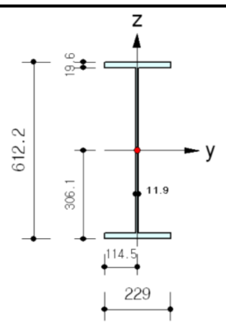
The simply supported 610 X 229 UB 125 of S275 steel shown in the right figure has a span of 6.0m and is laterally braced at 1.5m intervals. Check the adequacy of the beam for a nominal uniformly distributed dead load of 60 KNm together with a nominal uniformly distributed imposed load of 70 KNm.

3.15.1 Material Properties

Material	S275	f_y = 265 N/mm^2	E_s = 210 GPa

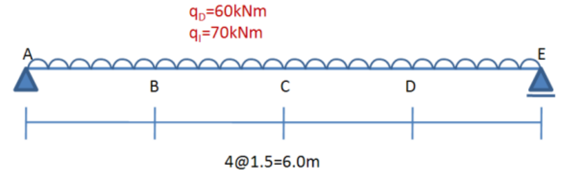
3.15.2 Section Properties

Section Name	UB 610x229x125
Depth *(H)*	612.2 mm
Width *(B)*	229.0 mm
Flange Thickness (T_f)	19.6 mm
Web Thickness (T_w)	11.9 mm
Gross sectional area *(A)*	15900.0 mm^2
Plastic section modulus ($W_{pl,y}$)	3676.0 cm^3



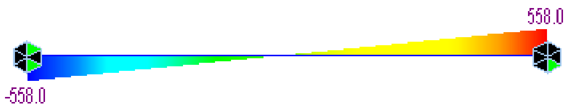
3.15.3 Analysis Model

Loading condition



Beam Diagram

SF



BM

3.15.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	1251.9kN	1171.00kN	6.46%
Bending resistance	975.2kNm	974.00kNm	0.12%

Note.
Shear resistance is calculated with an error of 6.46% due to the difference in shear area, Av. In the example book, the minimum value of shear area ‘$\eta h_w t_w$’ was not considered whereas it was considered in midas Gen. (6.2.6 (3) EN 1993-1-1:2005)

3.15.5 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.94 -. b/t = BTR = 4.89 -. sigma1 = 0.260 kN/mm^2. -. sigma2 = 0.260 kN/mm^2. -. BTR < 9*e **(Class 1 : Plastic).** **( ). Determine classification of bending Internal Parts**. [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.94 -. d/t = HTR = 46.02 -. sigma1 = 0.622 kN/mm^2. -. sigma2 = -0.622 kN/mm^2. -. HTR < 72*e **(Class 1 : Plastic).**	***Classifying the section.*** t_f = 19.6 mm, f_y = 265 N/mm^2 En 10025-2 $\varepsilon = \sqrt{(235/265)}^{0.5} = 0.942$ $c_f/(t_f\ \varepsilon)$ = (229/2 – 11.9/2 – 12.7) (19.6 × 0.942) = 5.19 < 9 and the flange is Class 1. $c_w/(t_w\ \varepsilon)$ = (612.2 – 2 ×19.6 - 2 ×12.7) / (11.9× 0.942) = 48.9 > 72 and the web is **Class 1.** (Note the general use of the minimum f_y obtained for the flange.)
2. Check Bending Moment Resistance About Major Axis **( ). Calculate plastic resistance moment about major axis**. [Eurocode3:05 6.1, 6.2.5] -. Wply = 3680000.0000 mm^3. -. Mc_Rdy = Wply * fy / Gamma_M0 =**975.20 kN-m**. **( ). Check ratio of moment resistance (M_Edy/Mc_Rdy).** M_Edy 837.0 -. ---------- = ----------- = 0.858 < 1.000 ---> O.K. Mc_Rdy 975.2	***Checking for moment***. q_{Ed} = (1.35× 60) + (1.5 × 70) = 186 KNm M_{Ed} = 186 × 6^2 /8 = 837 KNm $M_{c,Rd}$ = 3676 × 10^3 × 265 /1.0 Nmm = **975 KNm** > 837 KNm = M_{Ed} Which is satisfactory.
3. Check Shear Resistance. **(). Calculate shear area.** [Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2] -. eta = 1.2 (Fy < 460 MPa.) -. r = 12.7000 mm. -. Avy = Area - hw*tw = 9081.3000 mm^2. -. Avz1 = eta*hw*tw = **8182.4400** mm^2. -. Avz2 = Area - 2*B*tf + (tw + 2*r)*tf = **7654.2800** mm^2. -. Avz = MAX[Avz1, Avz2] = **8182.4400** mm^2.	***Checking for shear.*** $V_{Ed} = 186 \times \frac{6}{2} = 558$ KN A_v = 159X 10^2–2× 229 × 19.6 + (11.9 +2 × 12.7) × 19.6 = **7654** mm^2 $V_{c,Rd}$ = 7654 X (265 / √3) / 1.0 N = **1171 KN** > 558 KN = V_{Ed} Which is satisfactory.
( ). Calculate plastic shear resistance in local-z direction (Vpl_Rdz). *[Eurocode3:05 6.1, 6.2.6]* -. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0 = **1251.90**	***Checking for bending and shear.***

kN.

( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).
(LCB = 1, POS = J)

-. Applied shear force : V_Edz = 558.00 kN.

```
   V_Edz       558.00
-. ---------- = ------------ = 0.446 < 1.000 ---> O.K.
   Vpl_Rdz     1251.90
```

Note.
The difference in shear resistance occurred since the midas Gen consider the additional condition when calculating shear area as per EN1993-1-1:2005, sub clause 6.2.6(3) a).

$A_v = A - 2bt_f + (t_w + 2r)t_f$ **but not less than $\eta h_w t_w$**

The maximum M_{Ed} occurs at mid –span where V_{Ed} =0, and the maximum V_{Ed} occurs at the support where M_{Ed} = 0, and so there is no need to check for combined bending and shear. (Note that in any case, $0.5V_{c,Rd}$ = 0.5 × 1171 = 585.5 KN > 558 KN = V_{Ed} and so the combined bending and shear condition does not operate.)

[Reference]
N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 209-211 (Example 5.12.5)

3.16 Serviceability of a simply supported beam

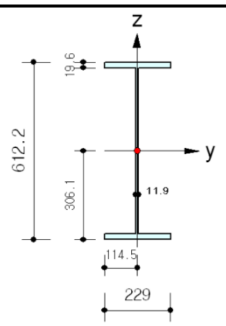
Check the imposed load deflection of the 610 X 229 UB 125 of right figure for a serviceability limit of L/360v.

3.16.1 Material Properties

Material	S275	f_y = 275 N/mm^2	E_s = 210 GPa

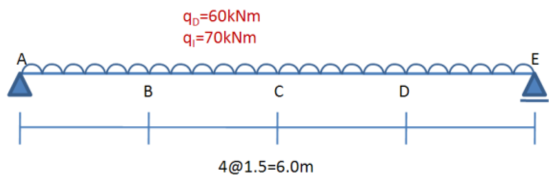
3.16.2 Section Properties

Section Name	UB 610x229x125
Depth *(H)*	612.2 mm
Width *(B)*	229.0 mm
Flange Thickness *(T_f)*	19.6 mm
Web Thickness *(T_w)*	11.9 mm
Gross sectional area *(A)*	15900.0 mm^2
Plastic section modulus ($W_{pl,y}$)	3676.0 cm^3



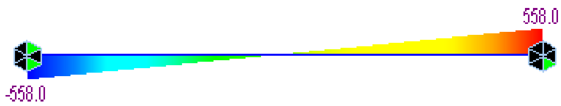
3.16.3 Analysis Model

Loading condition



Beam Diagram

SF



BM

3.16.4 Comparison of Design Results

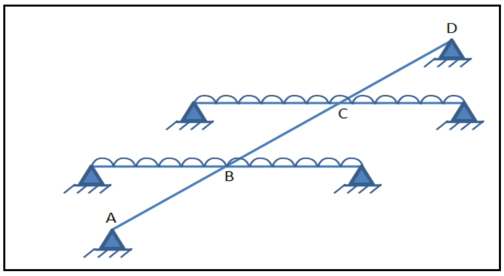
	midas Gen	Example book	Error (%)
Deflection (ω_c)	5.705mm	5.700mm	0.09%

3.16.5 Detailed comparison

midas Gen	Example book
1. Check Deflection. ***( ). Compute Maximum Deflection.*** -. LCB = 1 -. DAF = 1.000 (Deflection Amplification Factor). -. Position = 3000.000mm From i-end(Node 1). -. Def = -5.705 * DAF = -**5.705mm** (Golbal Z) -. Def_Lim = **16.667**mm Def < Def_Lim ---> O.K !	The central deflection ω_c of a simply supported beam with uniformly distributed load q can be calculated using $\omega_c = \frac{5qL^4}{384EI_y}$ $= \frac{5 \times 70 \times 6000^4}{384 \times 210000 \times 98610 \times 10^4}$ = **5.7 mm.** (The same result can be obtained using Figure 5.3.) L/360 =6000/360 = **16.7** mm > 5.7 mm = ω_c and so the beam is satisfactory. [Reference] N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 223 (Example 5.12.18)

3.17 Checking the major axis in-plane resistance

The 9 m long simply supported beam-column shown in Figure has a factored design axial compression force of 200 KN and a design concentrated load of 20 KN (which includes an allowance for self-weight) acting in the major principal plane at mid-span. The beam-column is the 254 X 146 UB 37 of S275 steel shown in Figure 7.19a. The become-column is continuously braced against lateral deflections v and twist rotations ϕ. Check the adequacy of the beam-column.

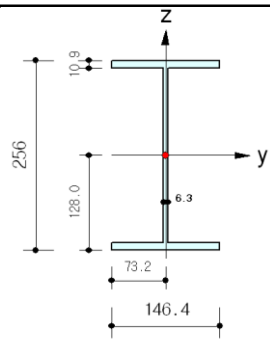


3.17.1 Material Properties

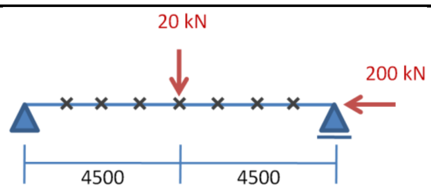
Material	S275	f_y = 275 N/mm^2	E_s = 210 GPa

3.17.2 Section Properties

Section Name	254x146 UB 37
Depth *(H)*	256.0 mm
Width *(B)*	146.4 mm
Flange Thickness *(T_f)*	10.9 mm
Web Thickness *(T_w)*	6.3 mm
Gross sectional area *(A)*	22000.0 mm^2
Shear area (A_{sz})	11500.2 mm^2



3.17.3 Analysis Model

Loading condition		
Beam Diagram	SF	
	BM	

3.17.4 Comparison of Design Results

	midas Gen	Example book	Error (%)
Shear resistance	900.13kN	900.00kN	0.01%
Bending resistance	132.82kNm	132.80kNm	0.02%

3.17.5 Detailed comparison

midas Gen	Example book
1. Cross-section classification **( ). Determine classification of compression outstand flanges.** [Eurocode3:05 Table 5.2 (Sheet 2 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. b/t = BTR = 5.73 -. sigma1 = 146298.978 KPa. -. sigma2 = 146298.978 KPa. -. BTR < 9*e (Class 1 : Plastic). **( ). Determine classification of bending and compression Internal Parts.** [Eurocode3:05 Table 5.2 (Sheet 1 of 3), EN 1993-1-5] -. e = SQRT(235/fy) = 0.92 -. d/t = HTR = 34.76 -. sigma1 = 197836.100 KPa. -. sigma2 = -113090.338 KPa. -. Psi = [2*(Nsd/A)*(1/fy)]-1 = -0.692 -. Alpha = 0.746 > 0.5 -. HTR < 396*e/(13*Alpha-1) (Class 1 : Plastic). ***2. CHECK AXIAL RESISTANCE*** **( ). Check slenderness ratio of axial compression member (Kl/i)** [Eurocode3:05 6.3.1] -. Kl/i = 83.3 < 200.0 ---> O.K. **( ). Calculate axial compressive resistance (Nc_Rd).** [Eurocode3:05 6.1, 6.2.4] -. Nc_Rd = fy * Area / Gamma_M0 = 1298.00 kN. **( ). Check ratio of axial resistance (N_Ed/Nc_Rd).** N_Ed 200.00 -. ----------- = ------------ = 0.154 < 1.000 ---> O.K. Nc_Rd 1298.00 **( ). Calculate buckling resistance of compression member (Nb_Rdy, Nb_Rdz).** [Eurocode3:05 6.3.1.1, 6.3.1.2] . Beta_A = Aeff / Area = 1.000 -. Lambda1 = Pi * SQRT(Es/fy) = 86.815 -. Lambda_by = {(KLy/iy)/Lambda1} * SQRT(Beta_A) = 0.960	***Simplified approach for cross-section resistance.*** t_f = 10.9 mm, f_y = 275 N/mm^2 En 10025-2 $\varepsilon = \sqrt{(235/275)}^{0.5} = 0.924$ $c_f/(t_f\,\varepsilon)$ = ((146.4 – 6.3 – 2 X 7.6) /2) (10.9 ×0.924) =6.20 <9 and the flange is **Class 1.** c_w = 256.0 – (2 X 10.9) – (2 X 7.6) = 219.0 mm *The compression proportion of web is* $\alpha = \left(\frac{h}{2} - (t_f + r) + \frac{1}{2}\frac{N_{Ed}}{t_f\,f_y}\right) / c_w$ $= \left(\frac{256}{2} - (10.9 + 7.6) + \frac{1}{2}\frac{200 \times 10^3}{6.3 \times 275}\right) / 219.0$ = 0.76 > 0.5 $c_w / t_w = 219.0 / 6.3 = 34.8 < 41.3 = \frac{396\varepsilon}{13\alpha - 1}$ and the web is **Class 1.** $M_{c,y,Rd}$ = 275 X 483 × 10^3 /1.0 Nmm =132.8 KNm $M_{y,Ed}$ = 20 X 9/4 =45.0 KNm $\frac{200 \times 10^3}{47.2 \times 10^2 \times 275/1.0} + \frac{45.0}{132.8} = 0.493 \le 1$ And the cross-section resistance is a adequate. Alternative approach for cross-section resistance. Because the section is Class 1, Clause 6.2.9.1 can be used. No reduction in plastic moment resistance is required provided both N_{Ed} = 200 KN < 324.5 KN = (0.25× 47.2 × 10^2 × 275/1.0)/ 10^3 = 0.25$N_{pl,Rd}$ and N_{Ed} = 200 KN < 202.9 KN $= \frac{0.5 \times (256.0 - 2 \times 10.9) \times 6.3 \times 275}{1.0 \times 10^3}$ $= \frac{0.5 h_w t_w f_y}{\gamma_{M0}}$ And so no reduction in the plastic moment resistance is required.

-. Ncry = Pi^2*Es*Ryy / KLy^2 = 1417.57 kN.
-. Lambda_by > 0.2 and N_Ed/Ncry > 0.04 --> Need to check.
-. Alphay = 0.210
-. Phiy = 0.5 * [1 + Alphay*(Lambda_by-0.2) + Lambda_by^2] = 1.040
-. Xiy = MIN [1 / [Phiy + SQRT(Phiy^2 - Lambda_by^2)], 1.0] = 0.693
-. Nb_Rdy = Xiy*Beta_A*Area*fy / Gamma_M1 =900.13 kN.
-. Lambda_bz = {(KLz/iz)/Lambda1} * SQRT(Beta_A) =3.310e-004
-. Ncrz = Pi^2*Es*Rzz / KLz^2 =11834642637.35 kN.
-. Lambda_bz < 0.2 or N_Ed/Ncrz < 0.04
--> No need to check.

( ). Check ratio of buckling resistance (N_Ed/Nb_Rd).

- Nb_Rd = MIN[Nb_Rdy, Nb_Rdz] = 900.13 kN.

```
     N_Ed        200.00
-. ----------- = -------------- = 0.222 < 1.000 ---> O.K.
     Nb_Rd       900.13
```

3. CHECK SHEAR RESISTANCE.

( ). Calculate shear area.

[Eurocode3:05 6.2.6, EN1993-1-5:04 5.1 NOTE 2]
-. eta = 1.2 (Fy < 460 MPa.)
-. r = 0.0076 m.
-. Avy = Area - hw*tw = 0.0032 m^2.
-. Avz1 = eta*hw*tw = 0.0018 m^2.
-. Avz2 = Area - 2*B*tf + (tw + 2*r)*tf = 0.0018 m^2.
-. Avz = MAX[Avz1, Avz2] = 0.0018 m^2.

( ). Calculate plastic shear resistance in local-z direction (Vpl_Rdz). [Eurocode3:05 6.1, 6.2.6]

-. Vpl_Rdz = [Avz*fy/SQRT(3)] / Gamma_M0 = 281.11 kN.

( ). Shear Buckling Check. [Eurocode3:05 6.2.6]

-. HTR < 72*e/Eta ---> No need to check!

( ). Check ratio of shear resistance (V_Edz/Vpl_Rdz).
(LCB = 1, POS = J)

-. Applied shear force : V_Edz = 10.00 kN.

```
     V_Edz      10.00
-. ------------ = ---------- = 0.036 < 1.000 ---> O.K.
     Vpl_Rdz    281.11
```

4. CHECK BENDING MOMENT RESISTANCE ABOUT MAJOR AXIS

( ). Calculate plastic resistance moment about major axis.

[Eurocode3:05 6.1, 6.2.5]
-. Wply = 0.0005 m^3.
-. Mc_Rdy = Wply * fy / Gamma_M0 = 132.82 kN-m.

Thus $M_{N,y,Rd} = M_{pl,y,Rd}$ = 132.9 KNm > 45.0 KNm $= M_{y,Ed}$

And the cross-section resistance is adequate.

Compression member buckling resistance.

Because the member is continuously braced, beam lateral buckling and column minor axis buckling need not be considered.

$\lambda_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \frac{L_{cr,y}}{i_y}\frac{1}{\lambda_1} = \frac{9000}{(10.8 \times 10)}\frac{1}{93.9 \times 0.924)}$ = 0.960

For a rolled UB section (with h/b > 1.2 and $t_f \leq$ 40 mm), buckling about the y-axis, use buckling curve (a) with α = 0.21

Φ_y = 0.5 [1 + 0.21 (0.960 – 0.2) + 0.960^2]= 1.041

$\chi_y = \frac{1}{1.041 + \sqrt{1.041^2 - 0.960^2}}$ = 0.693

$N_{b,y,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0.693 \times 47.2 \times 10^2 \times 275}{1.0}$ =900KN > 200KN = N_{Ed}

Beam-column member resistance – more exact approach(Annex A)

$\lambda_{max} = \lambda_y$ = 0.960

$N_{cr,y} = \frac{\pi^2 EI}{L_{cr}{}^2} = \frac{\pi^2 \times 210000 \times 55370000}{9000^2}$ = 1470 $\times$ 10^3
= 1470 KN

Since there is no lateral buckling, λ_0 = 0, b_{LT} = 0,

C_{mLT} = 1.0

$C_{my} = C_{my,0}$ = 1 – 0.18$N_{Ed}/N_{cr,y}$ =1 – 0.18$\times$ 200/ 1417
= 0.975

$W_y = \frac{W_{pl,y}}{W_{el,y}} = \frac{483}{433}$ = 1.115,

$n_{pl} = \frac{N_{ED}}{N_{RK}/\gamma_{M1})} = \frac{200 \times 10^3}{47.2 \times 10^2 \times 275/1.0}$ = 0.154

C_{yy} = 1 + (W_y − 1)

( ). Check ratio of moment resistance (M_Edy/Mc_Rdy).

```
         M_Edy        45.00
   -. -------------- = ----------- = 0.339 < 1.000 ---> O.K.
        Mc_Rdy        132.82
```

( ). Calculate plastic resistance moment about minor axis.

[Eurocode3:05 6.1, 6.2.5]

-. Wplz = 0.0001 m^3.

-. Mc_Rdz = Wplz * fy / Gamma_M0 = 32.73 kN-m.

( ). Check ratio of moment resistance (M_Edz/Mc_Rdz).

```
        M_Edz        0.00
  -. ------------- = --------- = 0.000 < 1.000 ---> O.K.
       Mc_Rdz        32.73
```

5. CHECK INTERACTION OF COMBINED RESISTANCE

( ). Calculate Major reduced design resistance of benging and shear. [Eurocode3:05 6.2.8 (6.30)]

-. In case of V_Edz / Vpl_Rdz < 0.5

-. My_Rd = Mc_Rdy = 132.82 kN-m.

( ). Calculate Minor reduced design resistance of benging and shear. [Eurocode3:05 6.2.8 (6.30)]

-. In case of V_Edy / Vpl_Rdy < 0.5

-. Mz_Rd = Mc_Rdz = 32.73 kN-m.

( ). Check general interaction ratio.

[Eurocode3:05 6.2.1 (6.2)] - Class1 or Class2

```
           N_Ed   M_Edy     M_Edz
 -. Rmax1 = ---------- + --------------- + ------------
           N_Rd   My_Rd     Mz_Rd
      = 0.493 < 1.000 ---> O.K.
```

( ). Check interaction ratio of bending and axial force member.

[Eurocode3:05 6.2.9 (6.31 ~ 6.41)] - Class1 or Class2

-. n = N_Ed / Npl_Rd = 0.154

-. a = MIN[(Area-2b*tf)/Area, 0.5] = 0.324

-. Alpha = 2.000

-. Beta = MAX[5*n, 1.0] = 1.000

-. N_Ed < 0.25*Npl_Rd = 225.03 kN.

-. N_Ed < 0.5*hw*tw*fy/Gamma_M0 = 202.88 kN.

Therefore, No allowance for the effect of axial force.

-. Mny_Rd = Mply_Rd = 132.82 kN-m.

-. Rmaxy = M_Edy / Mny_Rd = 0.339 < 1.000 ---> O.K.

-. N_Ed < hw*tw*fy/Gamma_M0 = 702.01 kN.

Therefore, No allowance for the effect of axial force.

-. Mnz_Rd = Mplz_Rd = 32.73 kN-m.

-. Rmaxz = M_Edz / Mnz_Rd = 0.000 < 1.000 ---> O.K.

-. Rmax2 = MAX[Rmaxy, Rmaxz] = 0.339 < 1.000 ---> O.K.

$$\left[\left(2 - \frac{1.6}{W_y} C_{my}{}^2 \lambda_{max} - \frac{1.6}{W_y} C_{my}{}^2 \lambda_{max}{}^2\right) n_{pl} - b_{LT} \geq W_{pl,y} \quad W_{el,y}\right.$$

$$= 1 + (1.115 - 1) \times \left(2 - \frac{1.6}{1.115} \times 0.975^2 \times 0.960 - \frac{1.6}{1.115} \times 0.975^2 \times 0.960^2\right) \times 0.154 - 0 = 0.990 > 0.896 = 1/1.115$$

$$\mu_y = \frac{1 - N_{ED}/N_{cr,y}}{1 - \chi_y N_{ED}/N_{cr,y}} = \frac{1 - 200/1417}{1 - 0.693 \times 200/1417} = 0.952$$

$$k_{yy} = C_{my} C_{mLT} \frac{\mu_y}{1 - N_{ED}/N_{cr,y}} \frac{1}{C_{yy}}$$

$$= 0.975 \times 1.0 \times \frac{0.952}{1 - 200/1417} \times \frac{1}{0.990} = 1.091$$

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{c,y,Rd}} = \frac{200}{900} + 1.091 \times \frac{45.0}{132.8}$$

$$= 0.222 + 0.370 = 0.592 < 1$$

And the member resistance is adequate.

( ). Check interaction ratio of bending and axial compression member.

[Eurocode3:05 6.3.1, 6.2.9.3 (6.61, 6.62), Annex A]

-. kyy = 1.091
-. kyz = 0.647
-. kzy = 0.645
-. kzz = 1.001
-. Xiy = 0.696
-. Xiz = 1.000
-. XiLT = 1.000
-. N_Rk = A*fy = 1298.00 kN.
-. My_Rk = Wply*fy = 132.82 kN-m.
-. Mz_Rk = Wplz*fy = 32.73 kN-m.
-. N_Ed*eNy = 0.0 (Not Slender)
-. N_Ed*eNZ = 0.0 (Not Slender)

```
                          N_Ed
-. Rmax_LT1  =   ----------------------------------
                  Xiy*N_Rk/Gamma_M1

                         M_Edy + N_Ed*eNy
            + kyy *  ------------------------------------
                       XiLT*My_Rk/Gamma_M1

                       M_Edz + N_Ed*eNz
            + kyz * ----------------------------------
                       Mz_Rk/Gamma_M1

            = 0.591  <  1.000  --->  O.K.


                          N_Ed
-. Rmax_LT2  =   ---------------------------------
                  Xiz*N_Rk/Gamma_M1

                         M_Edy + N_Ed*eNy
            + kzy * ------------------------------------
                       XiLT*My_Rk/Gamma_M1

                         M_Edz + N_Ed*eNz
            + kzz *  ----------------------------------
                         Mz_Rk/Gamma_M1
        = 0.373 < 1.000 ---> O.K.
```

-. Rmax = MAX[MAX(Rmax1, Rmax2), MAX(Rmax_LT1, Rmax_LT2)] = 0.591 < 1.000 ---> O.K.

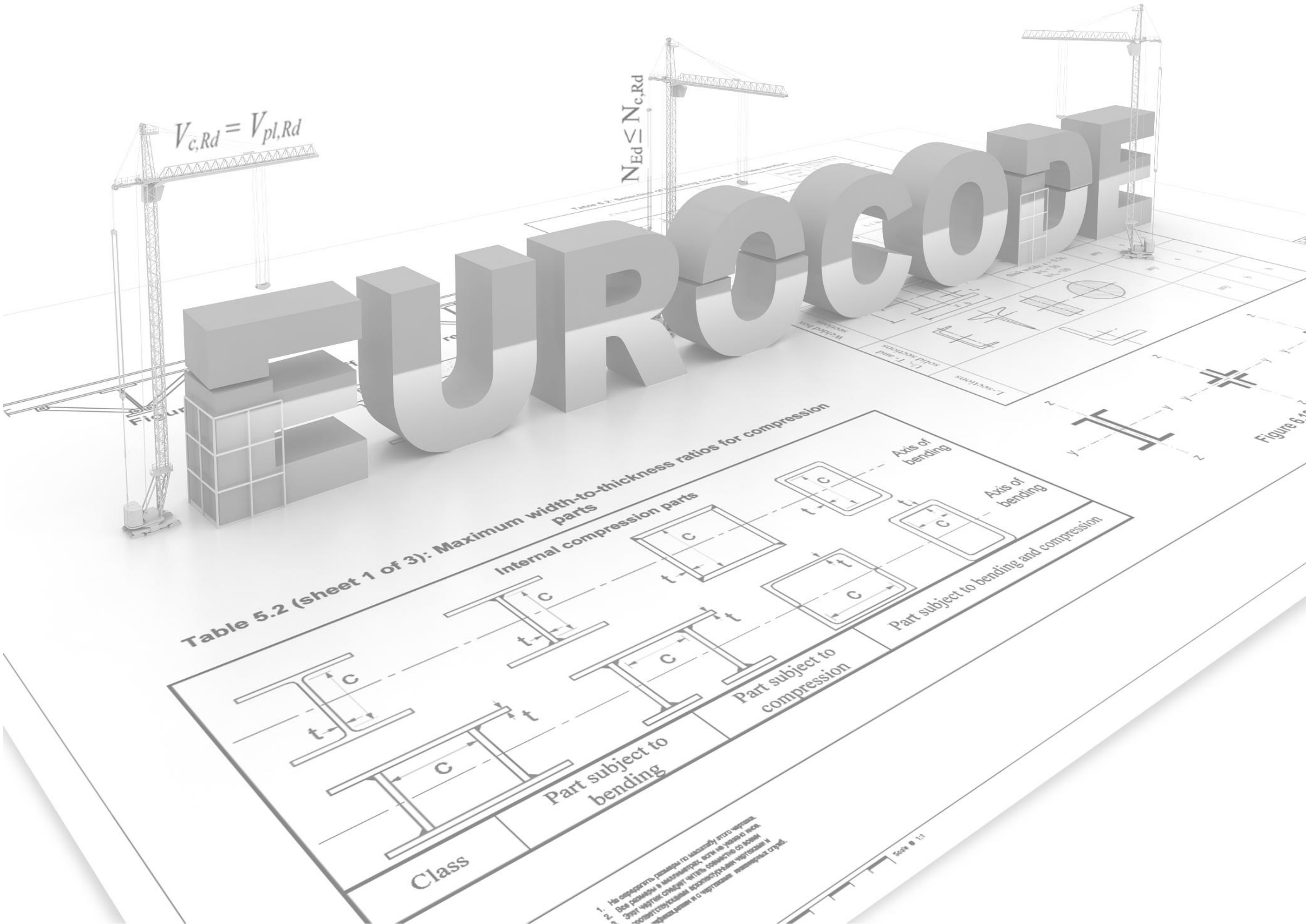
[Reference]

N.S. Trahair, M.A. Bradford, D.A.Nethercot, and L.Gardner, The behavior and Design of Steel Structures to EC3, Taylor & Francis, 330-333 (Example 7.7.2)

CHAPTER 4

Steel Design Tutorial

Design Examples using midas Gen to Eurocode3

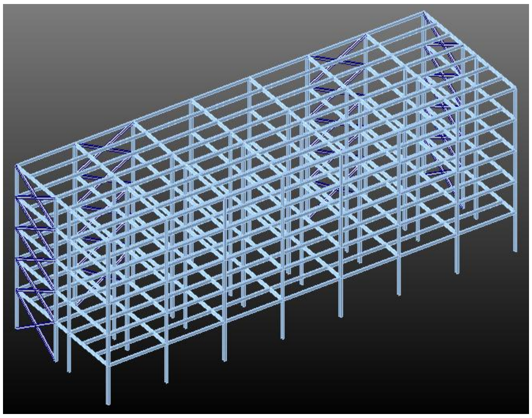


Step
00

Contents

Eurocode 3 - Design of Multi-Story Steel Building

- **Step 1:** Analyze the model.
- **Step 2:** Select the design code.
- **Step 3:** Generate load combinations.
- **Step 4:** Enter design parameters (Unbraced Length, Moment Factor, etc).
- **Step 5:** Enter deflection limits.
- **Step 6:** Check design results.
- **Step 7:** Change and update the designed sections.



Step
00

Overview

Eurocode 3 Steel Design Methods

midas Gen provides the following two methods:

1. The program finds optimal sections for gravity loads (Design > Steel Optimal Design) and also finds optimal sections for lateral loads (Design> Displacement Optimal Design). With the combined use of the two, the user should find optimal sections.

2. The program checks strength and serviceability based on the sections defined by the user and the design code selected by the user (Design > Steel Code Check). Also, the program searches and proposes sections which satisfy the design conditions entered by the user. Then the user can update the sections referring to the sections proposed by the program.

In this tutorial, method 2 is presented.

Step 00 *Overview* - Details of the example building

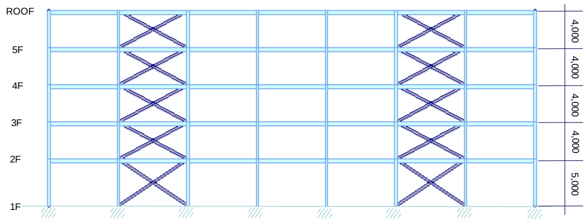


Figure 1. Elevation (unit: mm)

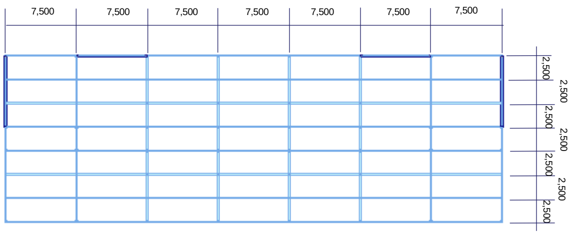


Figure 2. Structural Plan (2~Roof) (unit: mm)

Step 00 *Overview*

● **Applied Codes**

- Applied Wind Load: Eurocode 1 (2005)
- Applied Seismic Load: Eurocode 8 (2004)
- Steel Design Code: Eurocode 3 (2005)

● **Applied Loads**

Self Weight

Floor loads

• For floor 2~5

Superimposed Dead Load: 3.7 kN/m^2

Live Load: 4 kN/m^2

• For roof

Superimposed Dead Load: 5 kN/m^2

Live Load: 1.5kN/m^2

Wind loads

• Basic Wind Velocity: 26 m/s

• Terrain Category: II

Seismic loads

• Ground Type: B

● **Structural System**

- Bracing system

● **Materials**

- Beam, Column and Brace: S275

Unit Load Cases

Load		Name	Details
Static Load Cases	1	Self Weight	Self Weight
	2	SID	Superimposed Dead Load
	3	Live Load	Live Load
	4	Wind X-dir	Wind Load (in the global X-direction)
	5	Wind Y-dir	Wind Load (in the global Y-direction)
Response Spectrum Load Cases	6	RX	Seismic Load (in the global X-direction)
	7	RY	Seismic Load (in the global Y-direction)

Step
00 Overview

● **Applied Sections**

These are the sections assumed before design updates.

• **Beam**

Section ID	DB	Section Size
1	UNI	IPE 500
2	UNI	IPE 600
3	UNI	IPE 450

• **Column**

Section ID	DB	Section Size
4	UNI	HEB 240
5	UNI	HEB 300

• **Brace**

Section ID	DB	Section Size
6	UNI	HEA 260

Step
01 *Step . 1* Open the model file and perform analysis & Steel Design Code

Procedure

Step1. Open the model file and perform analysis

❶ Open "EC3 design_start model.mgb"

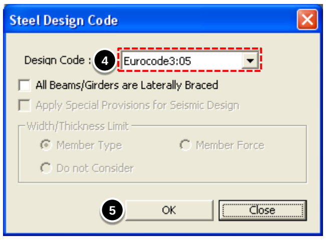
❷ Analysis > Perform Analysis

Step2. Steel Design Code

❸ Design > Steel Design Parameter > Design Code

❹ Design Code: "Eurocode3:05"

❺ Click on "OK" button.



Step **02** ***Step . 2* Generate Load Combinations**

Procedure

Generate Load Combinations

The program automatically creates design load combinations which can be also modified or deleted by the user.

❶ Result > Combinations

❷ Click on "Steel Design" Tab.

❸ Click "Auto Generation" button.

❹ Option: "Add"

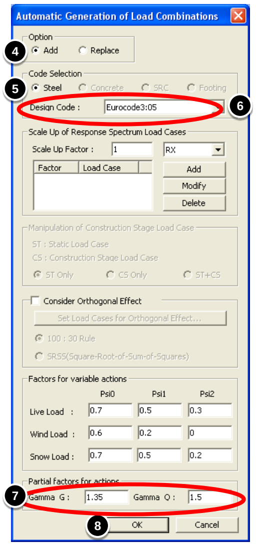
❺ Code Selection: "Steel"

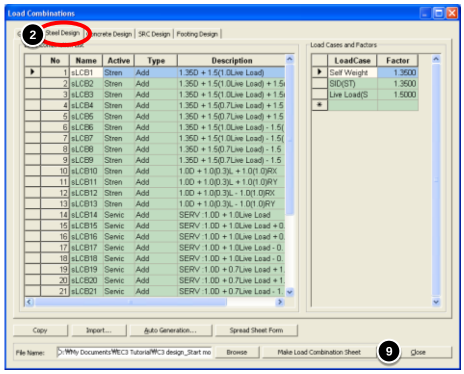
❻ Design Code: "Eurocode3:05"

❼ Gamma G: 1.35, Gamma Q: 1.5

❽ Click on "OK" button.

❾ Click on "Close" button.





Step **03** ***Step . 3* Enter Unbraced Length**

Procedure

Enter Unbraced Length

❶ View > Select > Identity

❷ Select Type: "Section"

❸ Select "1: IPE 500" & "IPE 600."

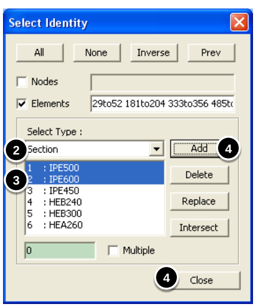
❹ Click on "Add" button and "Close" button.

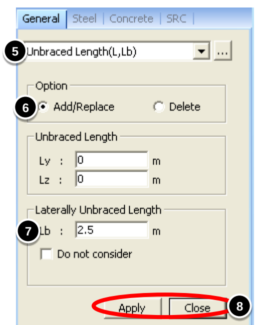
❺ Design > General Design Parameter >Unbraced Length

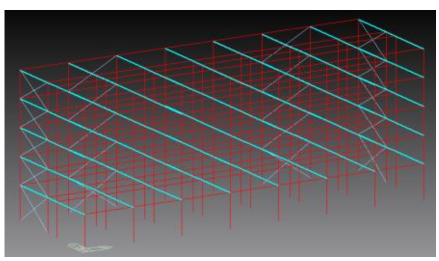
❻ Option: "Add/Replace"

❼ Laterally Unbraced Length, Lb = 2.5

❽ Click on "Apply" button and "Close" button.







Step 04

Step . 4 Enter Equivalent Uniform Moment Factor (Cmy, Cmz)

Procedure

Enter Equivalent Uniform Moment Factor (Cmy, Cmz)

❶ View > Select > Select All

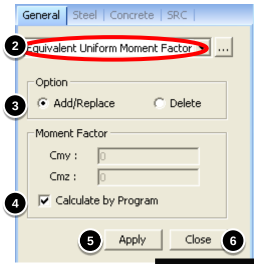
❷ Design > General Design Parameter >Equivalent Uniform Moment Factor

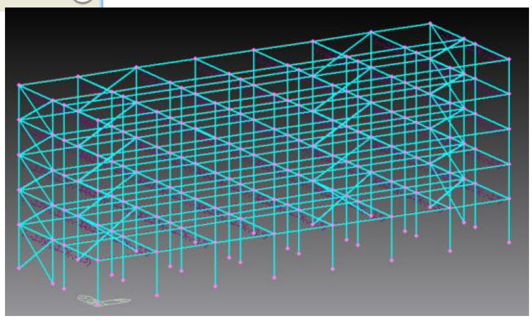
❸ Option: “Add/Replace”

❹ Check on “Calculate by Program”

❺ Click on “Apply” button

❻ Click on “Close” button.





Step 05

Step . 5 Enter Equivalent Moment Factor (CmLT)

Procedure

Enter Equivalent Moment Factor (CmLT)

❶ View > Select > Identity

❷ Select Type: “Element Type”

❸ Select “BEAM”.

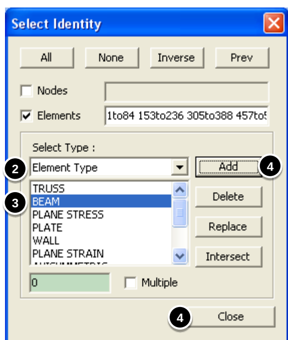
❹ Click on “Add” button and “Close” button.

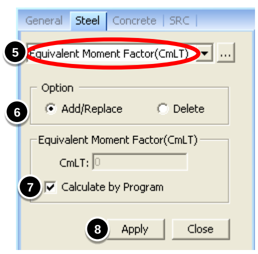
❺ Design > Steel Design Parameter > Equivalent Moment Factor

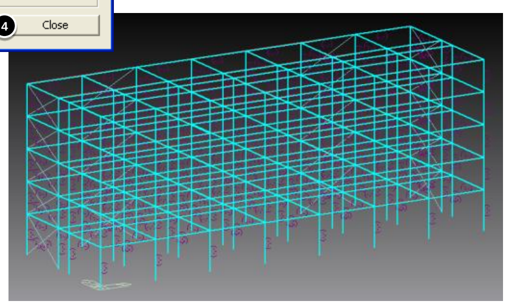
❻ Option: “Add/Replace”

❼ Check on “Calculate by Program.”

❽ Click on “Apply” button and “Close” button.







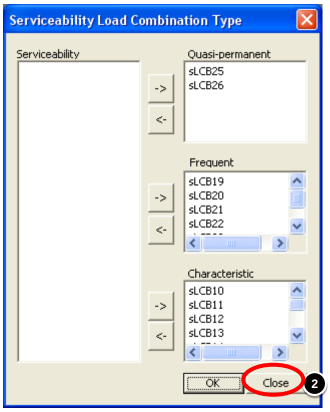
Step
06

Step . 6 Assign/Confirm Serviceability Load Combination Type

Procedure

Assign/Confirm Serviceability Load Combination Type

1. Design > General Design Parameter > Serviceability Load Combination Type
2. Click on "Close" button.



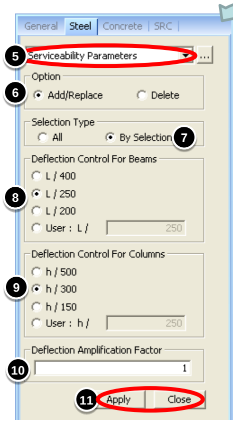
Step
07

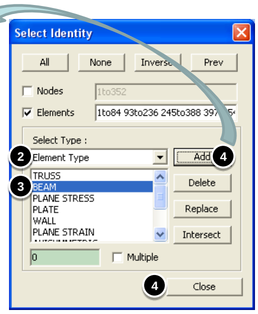
Step . 7 Enter Serviceability Parameters

Procedure

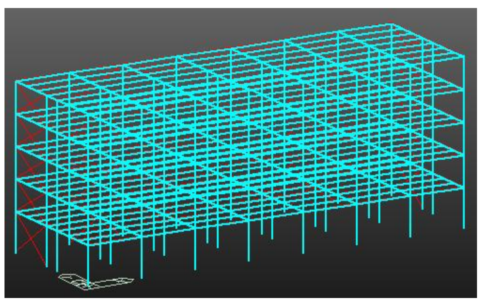
Enter Serviceability Parameters

1. View > Select > Identity
2. Select Type: "Element Type"
3. Select "BEAM."
4. Click on "Add" button and "Close" button.
5. Design > Steel Design Parameter > Serviceability Parameters
6. Option: "Add/Replace"
7. Selection Type: "By Selection"
8. Deflection Control For Beams: "L / 250"
9. Deflection Control For Columns: "h / 300"
10. Deflection Amplification Factor: "1"
11. Click on "Apply" button and "Close" button.





If the element's local x-axis is parallel to the global Z-axis, the element is considered as a column. If the element's local x-axis is parallel to global X-Y plane, the element is considered as beam. All other elements except for columns and beams are considered to be braces.



Step **08**

Step . 8-1 Steel Code Checking

Procedure

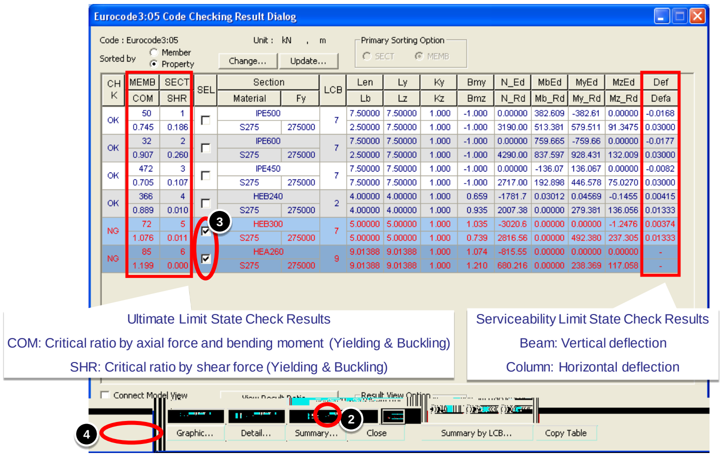
Steel Code Checking (1)

❶ Design > Steel Code Check

❷ Click on [<<] button.

❸ Select “SECT 5” & “SECT 6.”

❹ Click on “Graphic” button.



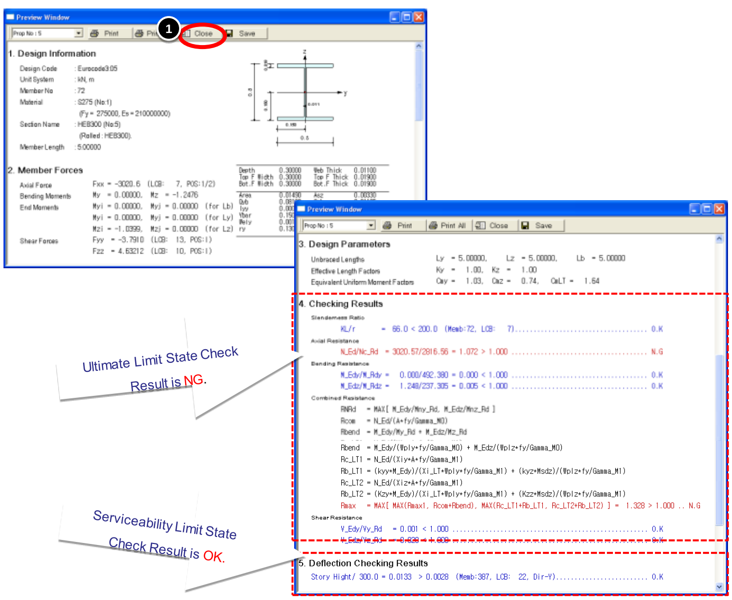
Step **08**

Step . 8-2 Steel Code Checking

Procedure

Steel Code Checking (2)

❶ Click on “Close” button.



Step
9

Step . 9-1 Change the NG sections

Procedure

Change the NG sections (1)

“Change” command will verify the strength for the user-selected section and save the design results until re-analysis is performed.

❶ Click on [>>] button.

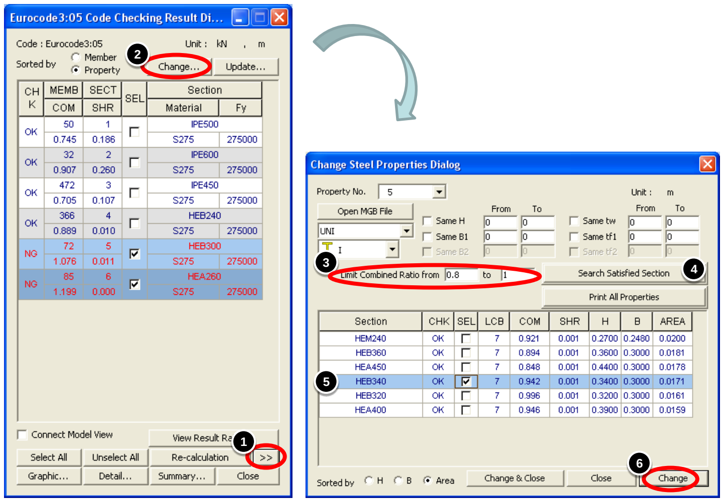
❷ Click on “Change” button.

❸ Limit Combined Ratio from “0.8” to “1.”

❹ Click on “Search Satisfied Section.”

❺ Select “HEB340.”

❻ Click on “Change” button.



Step
9

Step . 9-2 Change the NG sections

Procedure

Change the NG sections (2)

“Update” command will allow the user to update the section and re-analyze.

❶ Click on update button.

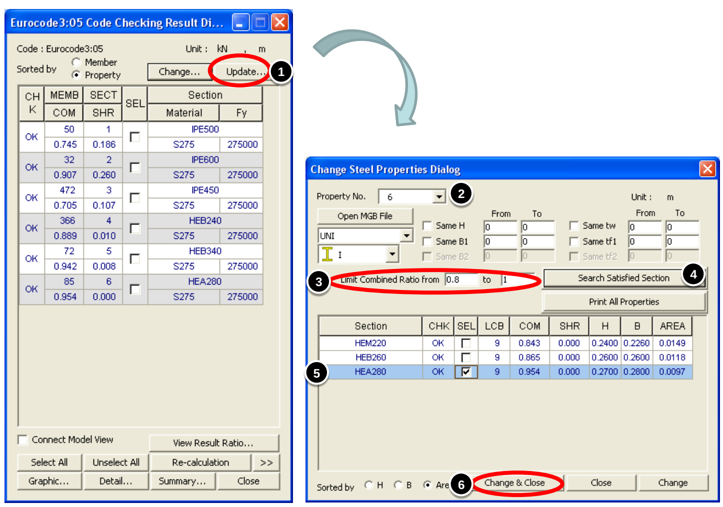
❷ Select Property No. 6.

❸ Limit Combined Ratio from “0.8” to “1.”

❹ Click on “Search Satisfied Section.”

❺ Select “HEA280.”

❻ Click on “Change & Close” button.



Step
9

Step . 9-3 Change the NG sections

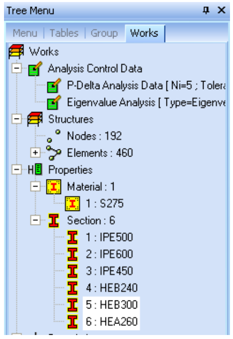
Procedure

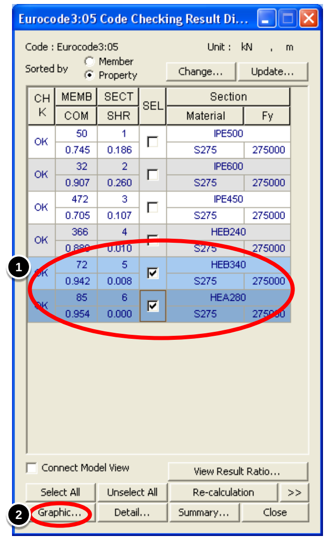
Change the NG sections (3)

Only the section for design review has been changed. The section in the model has not been changed as seen in the Works Tree.

❶ Select "SECT 6."

❷ Click on "Graphic" button.





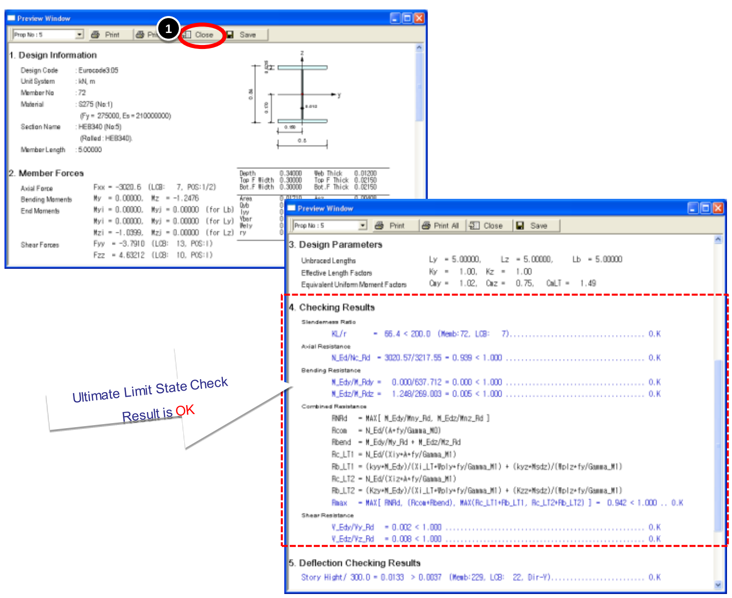
Step
9

Step . 9-4 Change the NG sections

Procedure

Change the NG sections (4)

❶ Click on "Close" button.



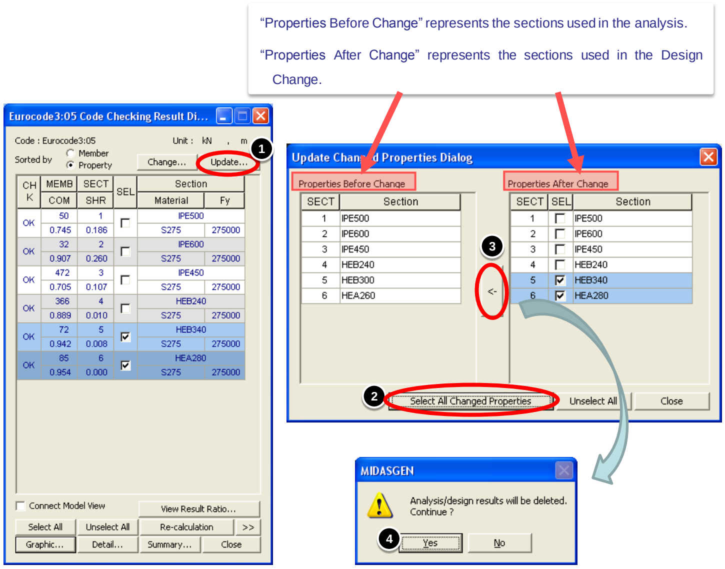
Step **10**

Step . 10-1 Updated the Design Sections

Procedure

Updated the Design Sections(1)

1. Click on “Update” button.
2. Click on “Select All Changed Properties” button.
3. Click on “<-” button.
4. Click on “Yes” button.



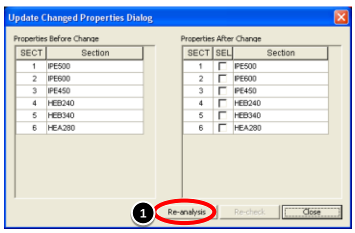
Step **10**

Step . 10-2 Updated the Design Sections

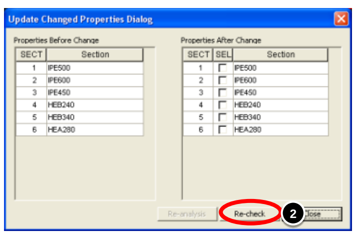
Procedure

Updated the Design Sections(2)

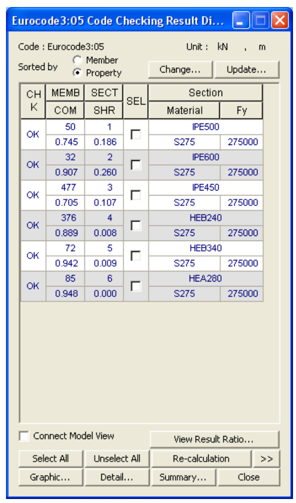
1. Click on “Re-analysis” button.
2. Click on “Re-check” button.



Re-analyze the model.



Re-do the steel code check.



Final design results

Step
11

Step . 11-1 Change the Section with Low Ratio

Procedure

Change the Section with Low Ratio (1)

❶ Click on “View Result Ratio” button.

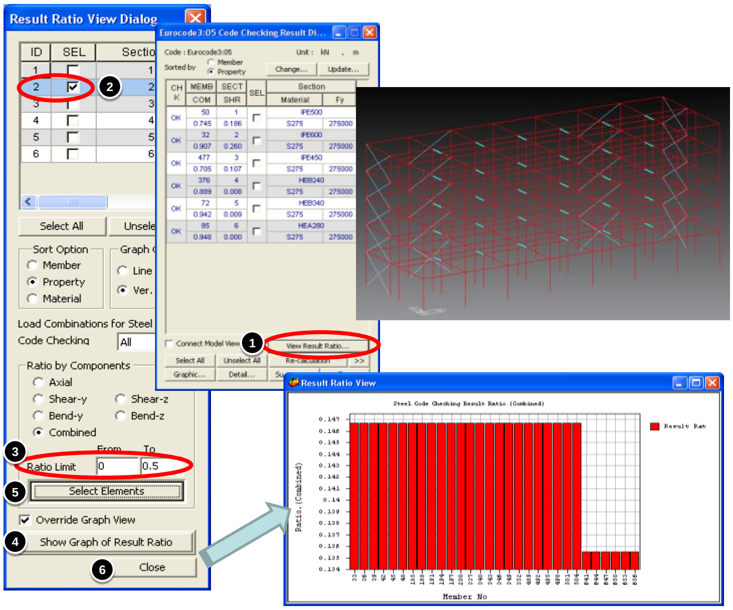
❷ Select “ID: 2.”

❸ Ratio Limit: From “0”, To “0.5”

❹ Click on “Show Graph of Result Ratio” button.

❺ Click on “Select Elements” button.

❻ Click on “Close” button.



Step
12

Step . 12-2 Change the Section with Low Ratio

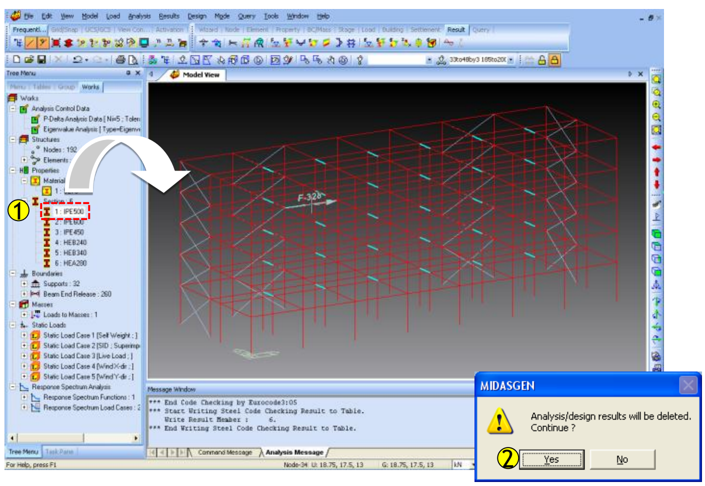
Procedure

Change the Section with Low Ratio (2)

Among the elements assigned with IPE600, the elements whose combined resistance ratio is less than 0.5 are changed into a smaller section IPE500.

❶ Drag & Drop “1: IPE500” into the Model View.

❷ Click on “Yes” button.



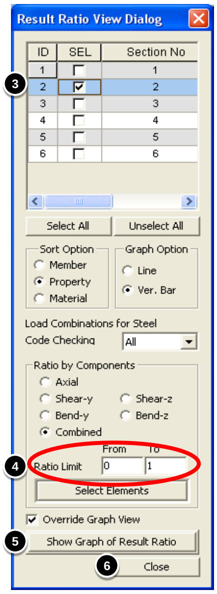
Step **12**

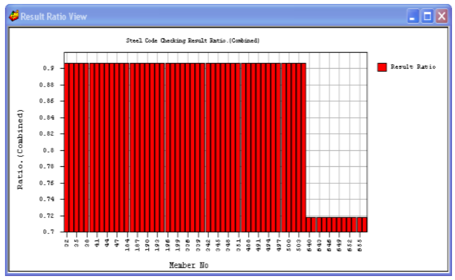
Step . 12-3 Change the Section with Low Ratio

Procedure

Change the Section with Low Ratio (3)

1. Analysis > Perform Analysis
2. Design > Steel Code Check
3. Click on "View Result Ratio" button.
4. Select "ID: 2."
5. Ratio Limit: From "0", To "1"
6. Click on "Show Graph of Result Ratio" button.
7. Click on "Close" button.





Combined resistance ratios of Section ID. 2 are all above 0.5.