DESIGN GUIDE
for midas Civil
Contents

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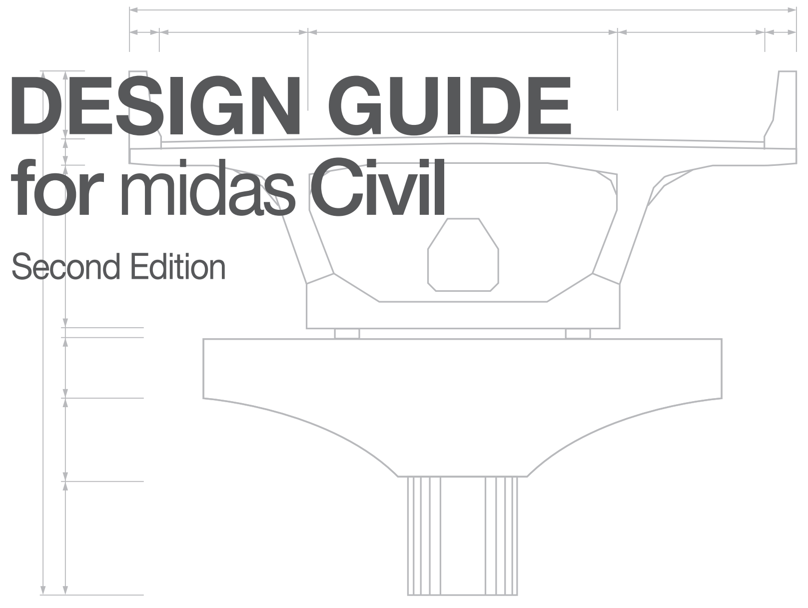
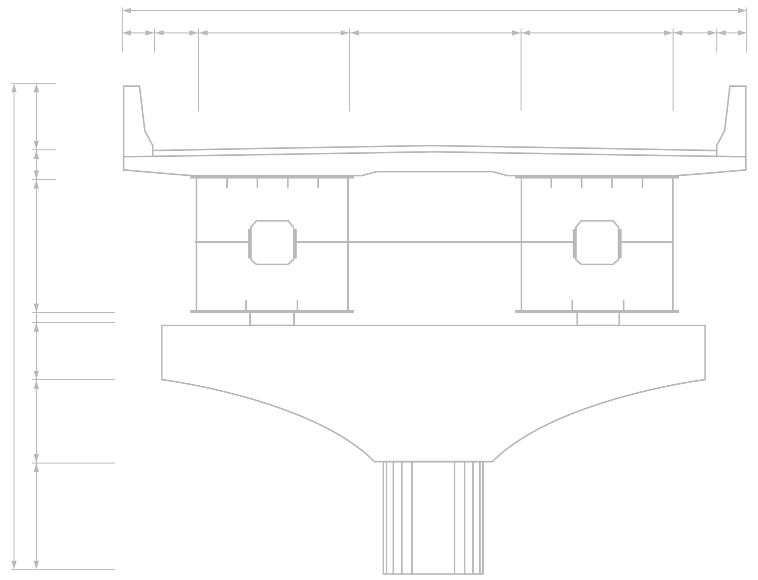
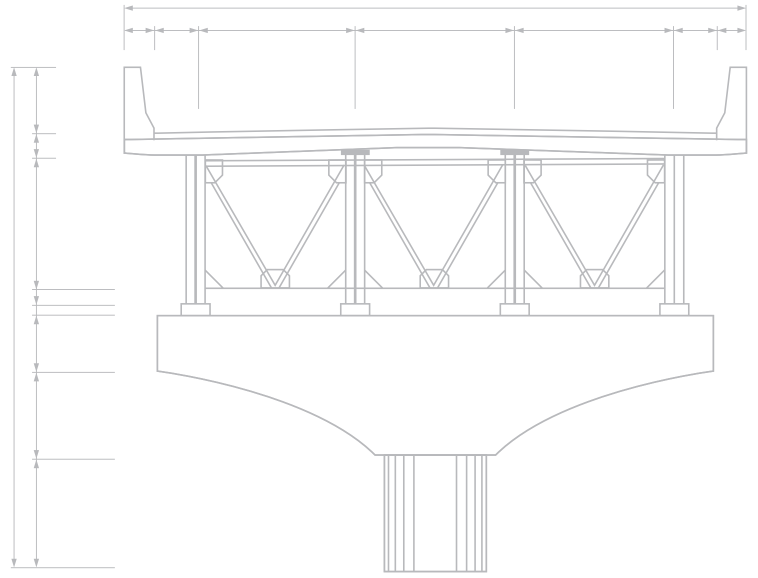
DESIGN GUIDE
for midas Civil
Second Edition



Prestressed Box Girder Design : EN 1992-2



Composite Steel Box Girder Design : EN 1994-2



Composite Plate Girder Design : EN 1994-2

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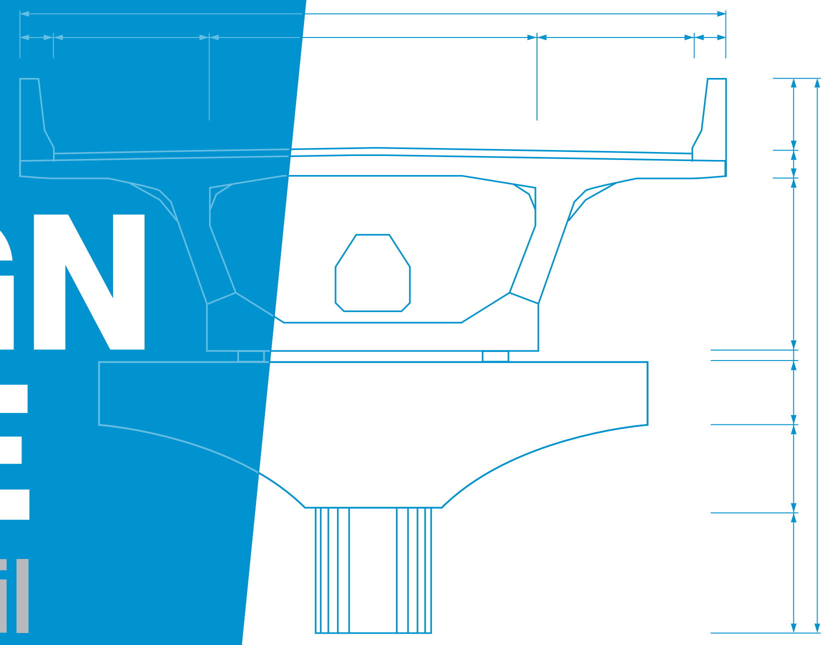
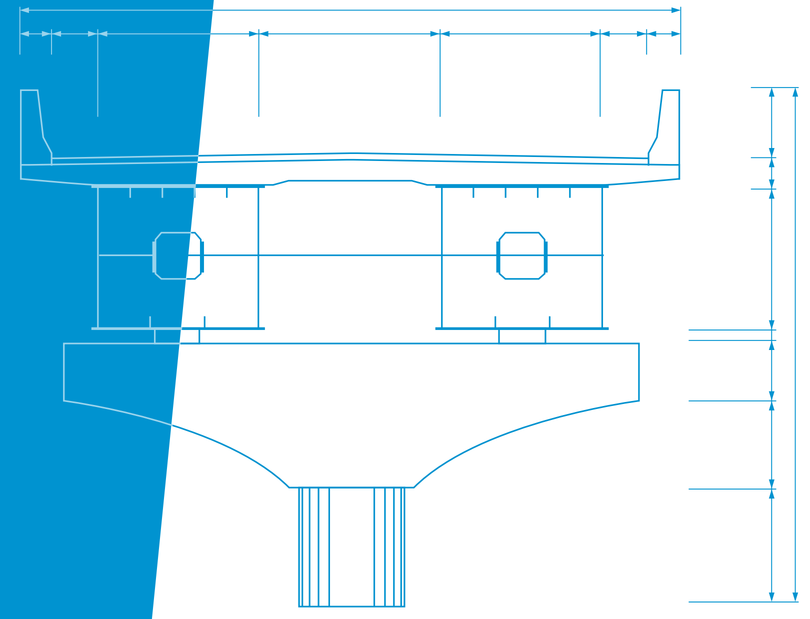
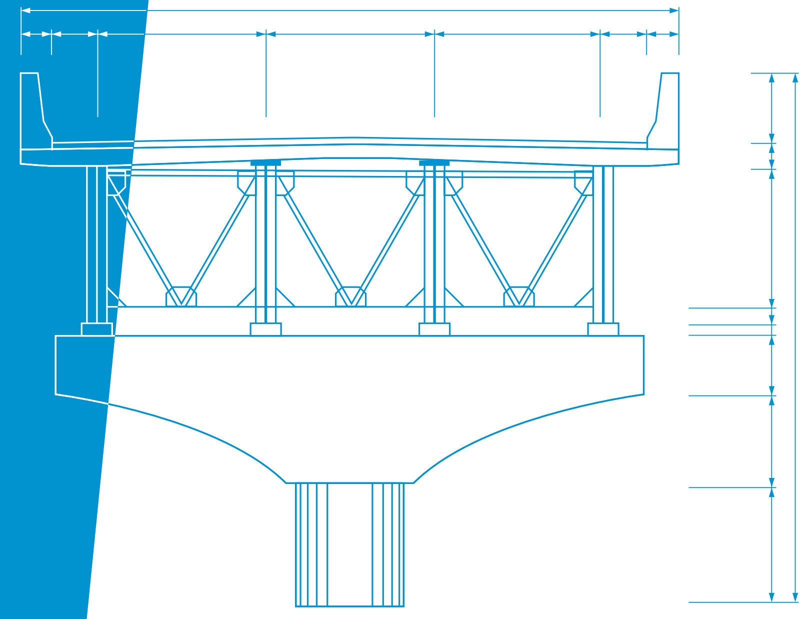
Prestressed Box Girder Design
Composite Steel Box Girder Design
Composite Plate Girder Design
Steel Frame Design
RC Frame Design







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The objective of this design guide is to outline the design algorithms which are applied in midas Civil finite element analysis and design system. The guide aims to provide sufficient information for the user to understand the scope, limitations and formulas applied in the design features and to provide relevant references to the clauses in the Design standards.

The design guide covers prestressed box girder, composite steel box girder, composite plate girder, steel frame and RC frame as per Eurocode.

It is recommended that you read this guide and review corresponding tutorials, which are found on our web site, http://www.MidasUser.com, before designing. Additional information can be found in the online help available in the program's main menu.

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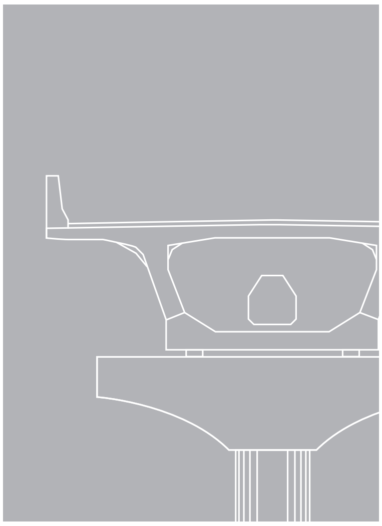
DISCLAIMER

Foreword



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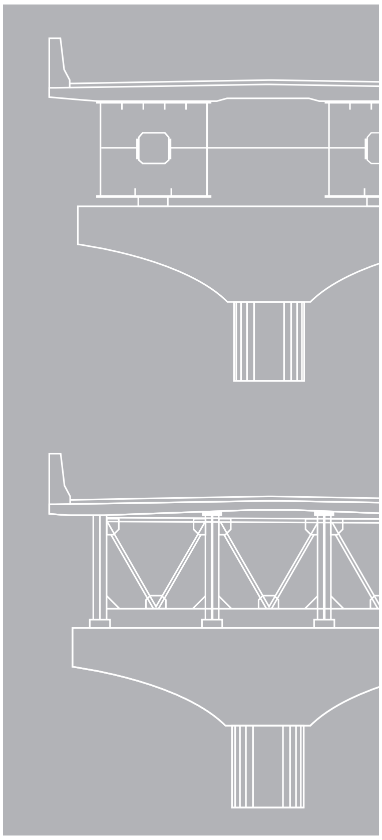
Organization



This guide is designed to help you quickly become productive with the design options of EN 1992-2, EN 1993-2 and EN 1994-2.

Chapter 1 provides detailed descriptions of the design parameters, ULS/SLS checks, design outputs used for prestressed box girder design to EN 1992-2.
Chapter 2 provides detailed descriptions of the design parameters, ULS/SLS checks, design outputs used for composite steel box girder design to EN 1994-2.
Chapter 3 provides detailed descriptions of the design parameters, ULS/SLS checks, design outputs used for composite plate girder design to EN 1994-2.
Chapter 4 provides detailed descriptions of the design parameters, ULS/SLS checks, design outputs used for steel frame design to EN 1993-2.
Chapter 5 provides detailed descriptions of the design parameters, ULS/SLS checks, design outputs used for RC frame design to EN 1992-2.

Although there is a huge overlap between Chapter 2 and Chapter 3 due to the similarity of structural types, the composite steel box girder and the composite plate girder are explained in two separate chapters for the convenience of the readers.

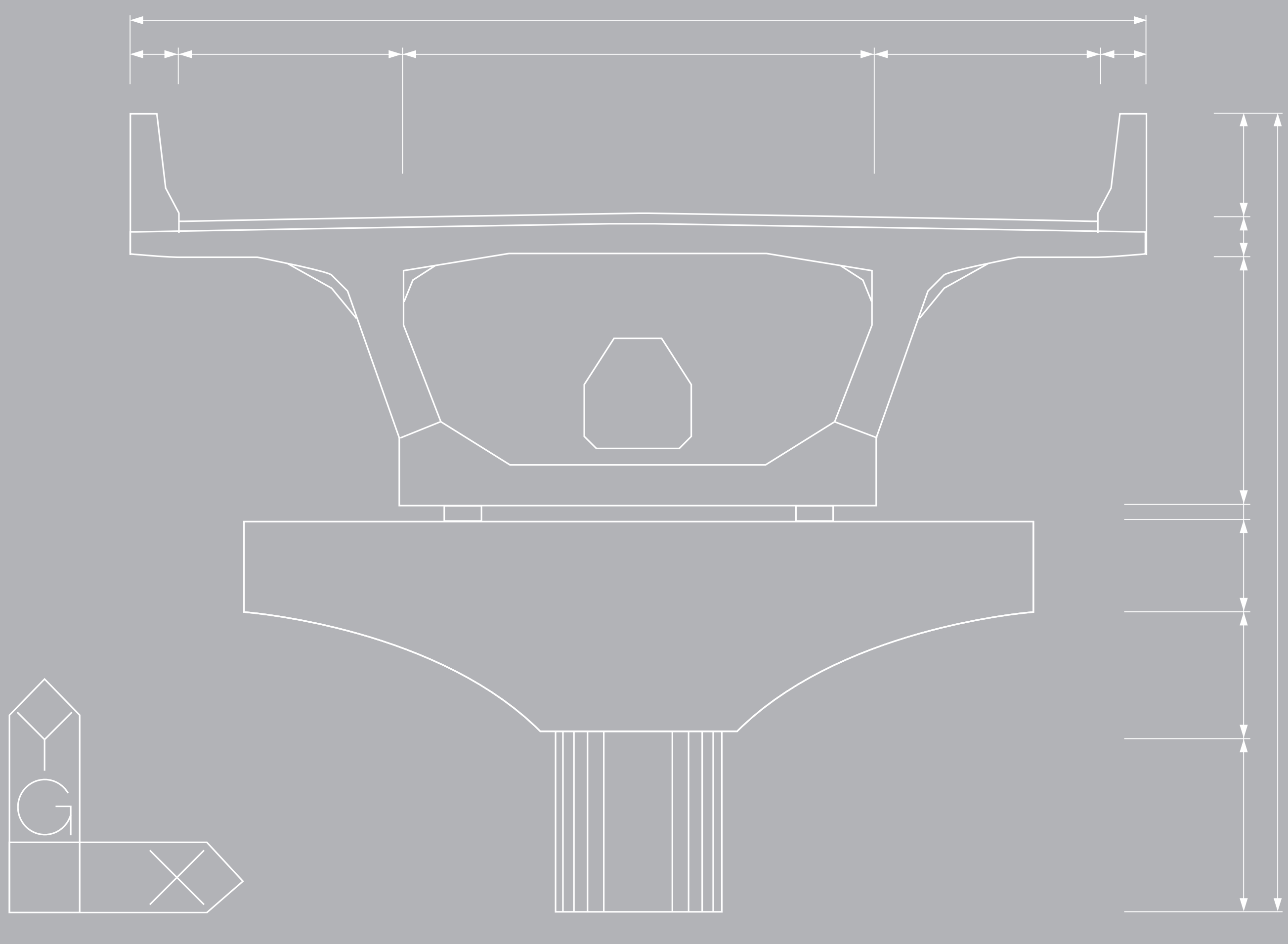
As the table of contents is printed on the folded flap, the readers can access the table of contents easily from any page of the book.

Contents

Chapter 1.

Prestressed Box Girder Design

EN 1992-2



Chapter 1.

Prestressed Box Girder Design (EN 1992-2)

Prestressed box girder needs to be designed to satisfy the following limit states.

Ultimate Limit States

- Moment Resistance
- Shear Resistance
- Torsion Resistance

Serviceability Limit States

- Stress for cross section at a construction stage
- Stress for cross section at service loads
- Tensile stress for Prestressing tendons
- Principal stress at a construction stage
- Principal stress at service loads
- Check crack width

Ultimate Limit States

1. Moment resistance

Limit state of moment resistance should satisfy the condition, $M_{Ed} \leq M_{Rd.}$

Moment resistance, M_{Rd}, is calculated using the strain compatibility method as shown below.

1.1 Design strength of material

(1) Design compressive strength of concrete

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \quad (1.1)$$

EN1992-1-1:2004
3.1.6(1)

where,

α_{cc}: The coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied.

f_{ck} : The characteristic compressive cylinder strength of concrete at 28 days.

γ_c : The partial safety factor for concrete.

(2) Design yield strength of reinforcement

$$f_{yd} = f_{yk} / \gamma_s \quad (1.2)$$

EN1992-1-1:2004
3.2.7(2)

where,

f_{yk}: The characteristic yield strength of reinforcement.

γ_s : The partial safety factor for reinforcement or prestressing steel.

(3) Design tensile strength of tendon.

$$f_{pd} = f_{p0,1k} / \gamma_s \quad (1.3)$$

EN1992-1-1:2004
3.3.6(6)

where,

$f_{p0,1k}$: The characteristic 0.1% proof-stress of prestressing steel.

γ_s : The partial safety factor for reinforcement or prestressing steel.

• Partial factors for materials γ_c, γ_s

If “User Input Data” option is checked on, the partial factors will be applied as the user defined value. However, if the option is checked off, the values in Table 1.1 will be applied.

[Table 1.1] Partial factors for materials for ULS

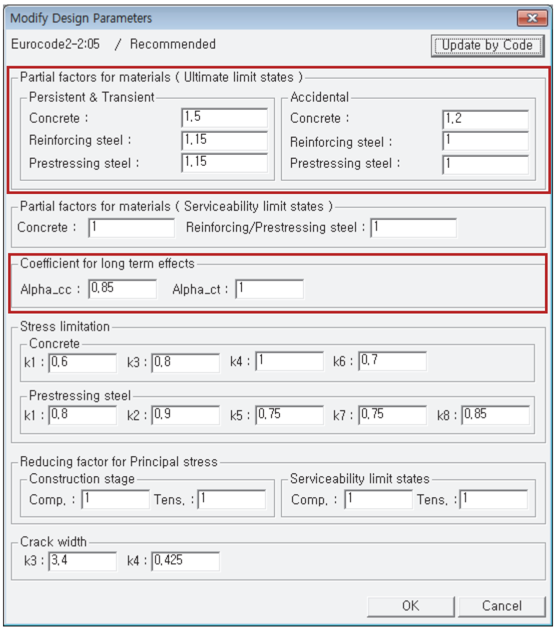
Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.5	1.15	1.15
Accidental	1.2	1.0	1.0

EN1992-1-1:2004
Table 2.1N

☐ Partial safety factor γ_c, γ_s / Coefficient for long term α_{cc}, α_{ct}

Main design parameters for materials can be entered in Modify Design Parameters dialog box. Among the input values, α_{cc} is considered when calculating moment resistance in Ultimate Limit State and it is applied as 1.0 for shear and torsional resistance.

Design > PSC Design > PSC Design Parameters…>Modify Design Parameters…



γ_c, γ_s

α_{cc}, α_{ct}

[Fig. 1.1] Modify Design Parameters Input Dialog

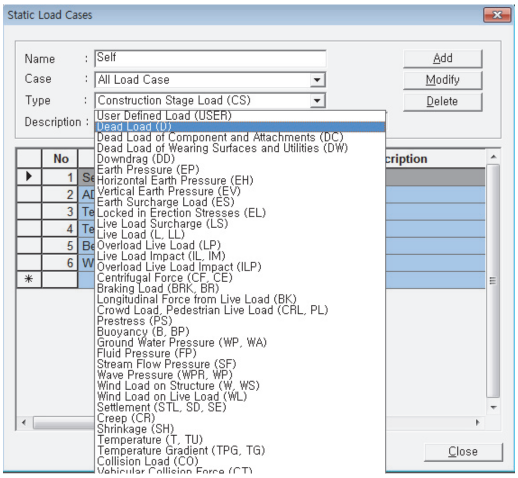
When defining partial factors for materials, Persistent & Transient and Accidental design situations can be specified as shown in Table 1.2.

[Table 1.2] Classification of design situations

Design situations	Description
Persistent & Transient	Load combination not "Accidental situation"
Accidental	Load combination include following load case type, Live Load Impact (IL, IM) Collision Load (CO) Vehicular Collision Force (CT) Vessel Collision Force (CV)

Load case type need to be specified in Static Load Cases dialog box.

Load>Static Load Cases…

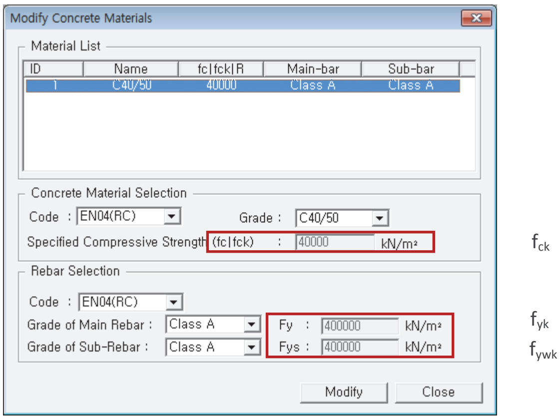


[Fig. 1.2] Static Load Cases Input Dialog

Strength of Concrete/Reinforcement

Define the material strengths of concrete and steel in PSC Design Material dialog box.

Design > PSC Design > PSC Design Material



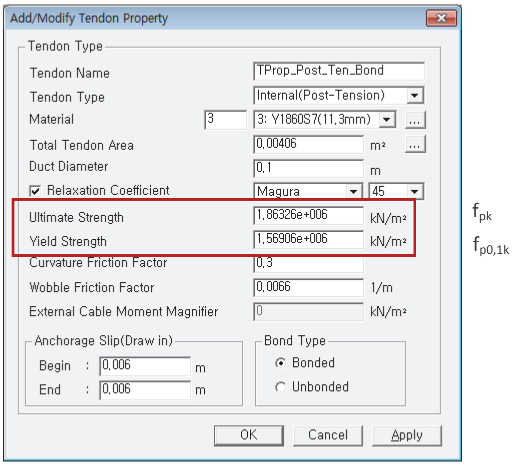
[Fig. 1.3] Define f_{ck}, f_{yk}, f_{ywk}

Select 'None' in the Code field and enter the name of the material to be used in the Name field. Then, each data field is activated and the strength of materials can be entered.

Strength of Tendon

Define the strength of tendon in Tendon Property dialog box.

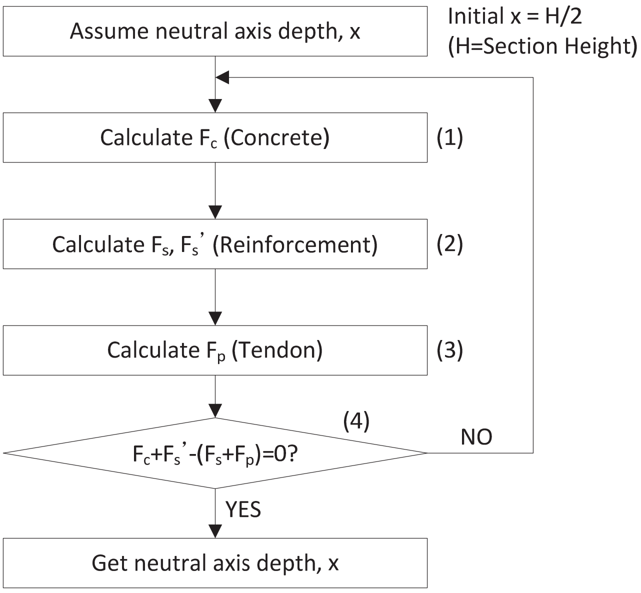
Load > Prestress Loads > Tendon Property



[Fig. 1.4] Define f_{pk}, $f_{p0.1k}$

1.2 Calculate neutral axis depth

Calculate the position of neutral axis by iterative approach as shown in the figure below.



[Fig. 1.5] Flow chart to calculate neutral axis depth, x

(1) Calculate force of concrete, F_c.

$$F_c = \eta f_{cd} \int_{dA} \lambda x \tag{1.4}$$

where,

λ: The effective height of the compression zone factor.

η: The effective strength factor.

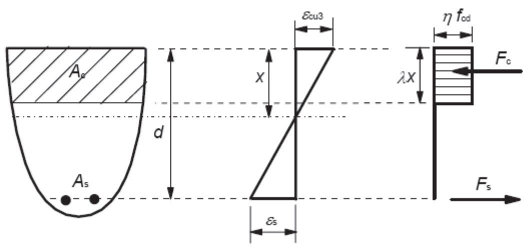
EN1992-1-1:2004 (3.19)~(3.22)

Condition	λ	η
f_{ck} ≤ 50MPa	0.8	1.0
50 < f_{ck} ≤ 90MPa	0.8-(f_{ck}-50)/400	1.0-(f_{ck}-50)/200
f_{ck} > 90MPa	0.7	0.8

x: The neutral axis depth.

• In midas Civil, a rectangular stress distribution is used as shown in the figure below.
(Ultimate strain of concrete $\varepsilon_{cu} = \varepsilon_{cu3}$)

EN1992-1-1:2004 Figure 3.5



[Fig. 1.6] Rectangular stress distribution

(2) Calculate force of reinforcement, F_s, F_s'.

$$F_s = A_s f_s, \quad F_s' = A_s' f_s' \tag{1.5}$$

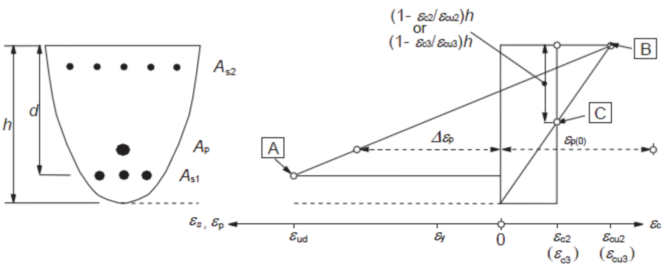
where,
A_s, A_s': The cross sectional area of tensile and compressive reinforcement.
f_s, f_s': The stress of tensile and compressive reinforcement.

In order to calculate the stress of reinforcing steel, f_s and f_s', calculate the appropriate strain by the strain compatibility condition. And then calculate the corresponding stresses in the stress-strain diagram.
Calculation method of strain and stress is as follow.

• Calculate the strains of reinforcement by assuming a linear strain distribution and the strain of ε_{cu3} at the extreme fiber of the concrete in compression.

$$\varepsilon_s = \frac{d_t - x}{x}\varepsilon_{cu}, \quad \varepsilon_s' = \frac{x - d_c}{x}\varepsilon_{cu} \tag{1.6}$$

where,
ε_s : The strain of tensile reinforcement.
ε_s': The strain of compressive reinforcement.
ε_{cu}: The ultimate compressive strain in the concrete. ($\varepsilon_{cu} = \varepsilon_{cu3}$)
x : The neutral axis depth.
d_t : Distance from the tensile rebar to the extreme top fiber of the element
d_c : Distance from the compressive rebar to the extreme top fiber of the element



[Fig. 1.7] Possible strain distributions in the ultimate limit state

EN1992-1-1:2004
Figure 6.1

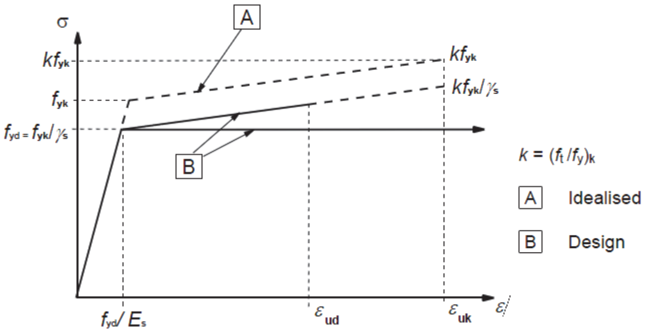
• Calculate the reinforcement stresses appropriate to the calculated reinforcement strains. (from the stress-strain idealizations)

$$f_s = \begin{cases} \varepsilon_s E_s & (\varepsilon_s \le \varepsilon_{yd}) \\ f_{yd} & (\varepsilon_s > \varepsilon_{yd}) \end{cases}, \quad f_s' = \begin{cases} \varepsilon_s' E_s & (\varepsilon_s' \le \varepsilon_{yd}) \\ f_{yd} & (\varepsilon_s' > \varepsilon_{yd}) \end{cases} \tag{1.7}$$

$$\varepsilon_{yd} = f_{yd} / E_s \tag{1.8}$$

$$f_{yd} = f_{yk} / \gamma_s \tag{1.9}$$

where,
E_s *: The design value of modulus of elasticity of reinforcement.*
f_{yd} *: The design yield strength of reinforcement. (See 1.1(2))*
ε_{yd} *: The yield strain of reinforcement.*



[Fig. 1.8] Idealized and design stress-strain diagram for reinforcing steel

EN1992-1-1:2004
Figure 3.8

(3) Calculate force of tendon, F_p.

$$F_p = \sum A_{pi} f_{pi} \tag{1.9}$$

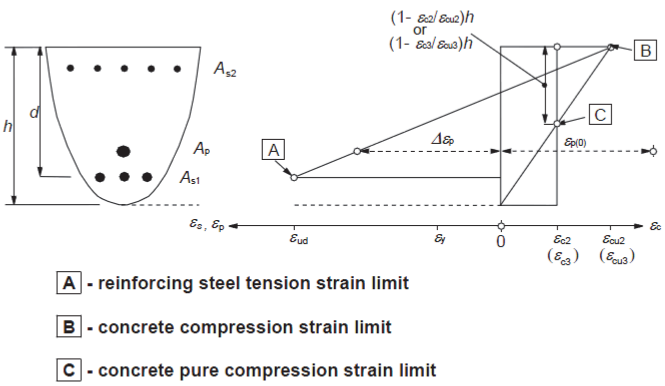
where,
A_{pi} *: The cross sectional area of tendon.*
f_{pi} *: The stress of tendon.*

In order to calculate the stress of tendon, f_{pi}, calculate the appropriate strain by the strain compatibility condition. And then calculate the corresponding stresses in the stress-strain diagram.
Calculation method of strain and stress is as follow.

• Calculate the strains of reinforcement by assuming a linear strain distribution and the strain of ε_{cu3} at the extreme fiber of the concrete in compression.

$$\varepsilon_p = \Delta\varepsilon_p + \varepsilon_{p(0)} = \varepsilon_{cu}\frac{d-x}{x} + \frac{\sigma_{eff}}{E_p} \tag{1.10}$$

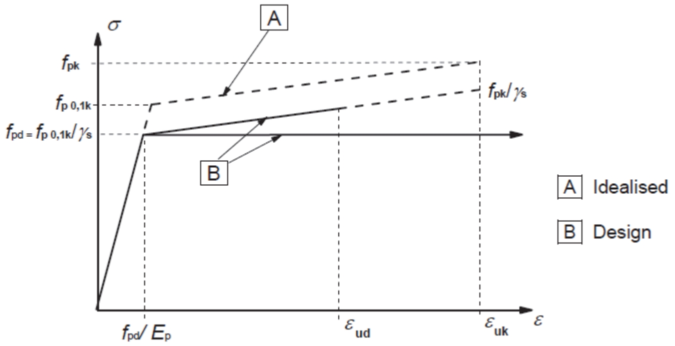
where,
$\Delta\varepsilon_p$ *: The change in strain in prestressing steel.*
$\varepsilon_{p(0)}$*: The initial strain in prestressing steel.*
σ_{eff} *: The stress under the effective prestress,* P_e
E_p *: The design value of modulus of elasticity of prestressing steel.*



[Fig. 1.9] Possible strain distributions in the ultimate limit state

EN1992-1-1:2004
Figure 6.1

• Calculate the reinforcement stresses appropriate to the calculated reinforcement strains. (from the stress-strain idealizations)



EN1992-1-1:2004
Figure 3.10

[Fig. 1.10] Idealized and design stress-strain diagrams for prestressing steel

EN1992-1-1:2004
5.10.8(2)

[Table 1.3] Stress of tendon, f_p

Tendon Type	Bond Type	f_p
Internal (Pre-tension)	Bonded	$f_p = \varepsilon_p E_p = (\Delta\varepsilon_p + \varepsilon_{p(0)})E_p \le f_{pd}$
Internal (Post-tension)	Bonded	$f_p = \varepsilon_p E_p = (\Delta\varepsilon_p + \varepsilon_{p(0)})E_p \le f_{pd}$
	Unbonded	$f_p = f_{pe} + \Delta\sigma_{p,ULS} = f_{pe} + 100MPa$
External	Unbonded	$f_p = f_{pe} + \Delta\sigma_{p,ULS} = f_{pe} + 100MPa$

where,

$\Delta\sigma_{p,ULS}$: The increase of the stress from the effective prestress to the stress in the ultimate limit state. The recommended value is 100MPa.

(4) Check if resultant force is zero.

Determine the neutral axis position by iterative approach of the clause (1) to (3) until the compressive strength ($C=F_c+F_s'$) and tensile strength ($T=F_s+F_p$) become identical. In midas Civil, convergence condition for "C = T" is applied as follows.

• Convergence condition :

$$\left|\frac{C}{T} - 1.0\right| < 0.001 \qquad (Tolerance) \tag{1.11}$$

where,

$$C = F_c + F_s', \quad T = F_s + F_p \tag{1.12}$$

• Reassume neutral axis depth by "Bisection method (Numerical analysis)" before meet following stop condition.

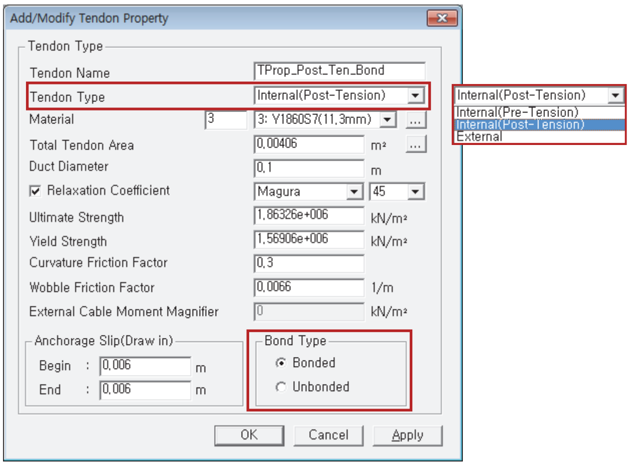
[Table 1.4] Stop condition for iterative approach

Stop condition	Description
Converge	$\frac{C}{T} - 1.0 < 0.001$
Not converge	Repeat count > 20 → Output "Not converge" in Message window. → Need to modify model as following. - Increase section size. - Modify prestressing forces in tendons. - Modify the position of tendons.

□ Tendon Type

Define the tendon type and bond type in Tendon Property dialog box.

Load > Prestress Loads > Tendon Property



[Fig. 1.11] Tendon Property Input Dialog

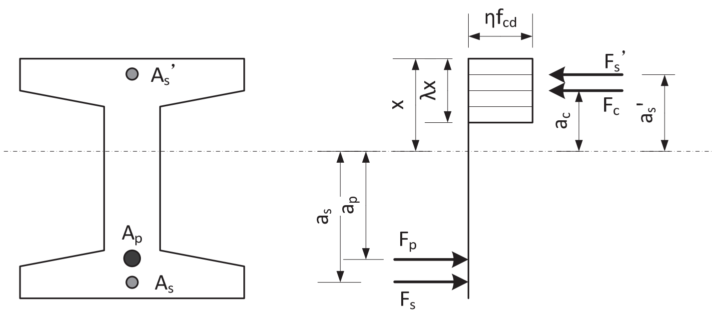
1.3 Calculate moment resistance M_{Rd}

Once the neutral axis is calculated, moment resistance can be calculated by multiplying the axial forces and eccentricity from the neutral axis.

$$M_{Rd} = F_c a_c + F_s' a_s' + F_s a_s + \sum \left(F_{pi} a_{pi} \right) \tag{1.13}$$

where,

a_c, a_s, a_s', a_{pi} : *The distance from neutral axis depth, x to concrete, reinforcement rebar, tendon.*



[Fig. 1.12] Forces and distances from neutral axis depth for M_{Rd}

1.4 Check moment resistance

$$M_{Ed} \le M_{Rd} \tag{1.14}$$

where,

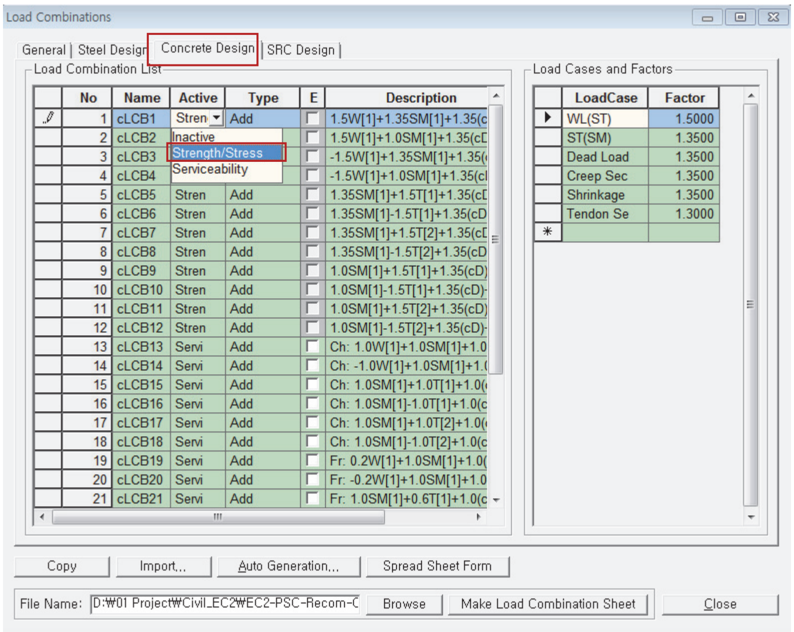
M_{Ed} : *Design value of the applied internal bending moment.*

M_{Rd} : *Design moment resistance.*

• Design load combination

In midas Civil, load combination to be used in PSC design can be defined in Results>Load combinations>Concrete Design tab. Moment resistance is verified with the most critical positive and negative design moment among the load combinations specified as " Strength/Stress" in Active column.

Results>Load Combinations...



[Fig. 1.13] Load Combinations Input Dialog

1.5 Verification of moment resistance

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check Flexural Strength...

Model View / Check Flexure Strength

Elem	Part	Positive/ Negative	LCom Name	Design Situations	Type	CHK	M_Ed (kN·m)	M_Rd (kN·m)	M_Ed/M_Rd	Aps (m²)
1	I[1]	Negative	cLCB3	Persistent &	FX-MAX	OK	0.0000	11363.7864	0.0000	0.0162
1	I[1]	Positive	cLCB7	Persistent &	FX-MIN	OK	49.8349	9483.2365	0.0053	0.0162
1	J[2]	Negative	cLCB1	Persistent &	FX-MIN	OK	0.0000	12440.4492	0.0000	0.0162
1	J[2]	Positive	cLCB7	Persistent &	FX-MAX	OK	1556.1181	12359.1870	0.1259	0.0162
10	I[10]	Negative	cLCB1	Persistent &	FX-MIN	OK	-5109.7398	13809.4738	-0.3700	0.0162
10	I[10]	Positive	cLCB7	Persistent &	FX-MAX	OK	4937.2958	21750.3938	0.2270	0.0162
10	J[11]	Negative	cLCB1	Persistent &	FX-MIN	OK	-6690.9174	17046.3183	-0.3925	0.0162
10	J[11]	Positive	cLCB7	Persistent &	FX-MAX	OK	4472.4522	19225.4072	0.2326	0.0162

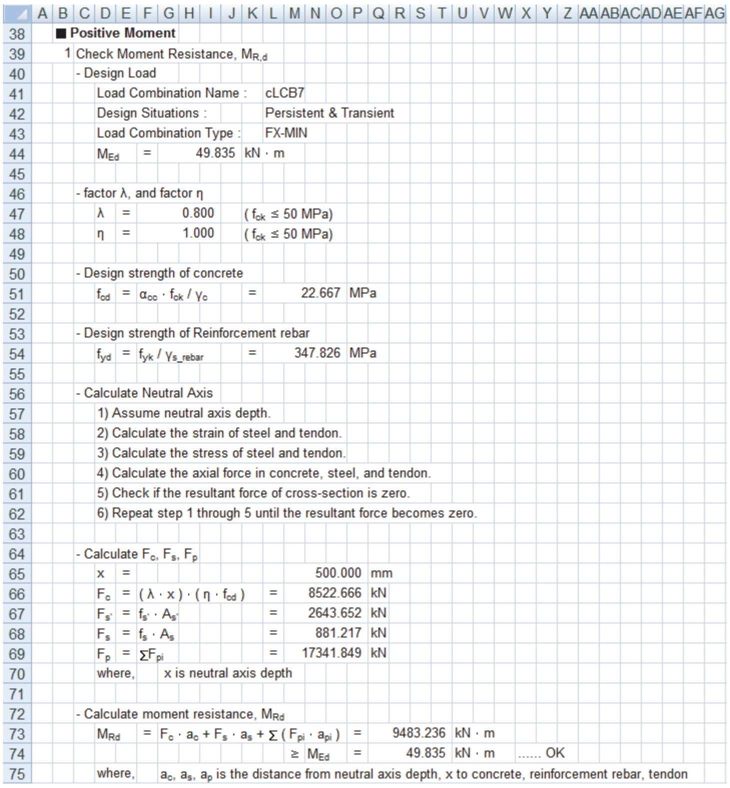
[Fig. 1.14] Result table for moment resistance

Elem : Element number
Part : Check location (I-End, J-End) of each element.
Positive/Negative : Positive moment, negative moment.
LCom Name : Load combination name.
Type : Displays the set of member forces corresponding to moving load case or settlement load case for which the maximum stresses are produced.
CHK : Flexural strength check for element
M_Ed : Design moment
M_Rd : Moment resistance.
M_Ed/M_Rd : The ratio of design moment to moment resistance.
Aps: Cross sectional area of tensile tendons.

By Excel Report

Detail design results including applied equations and design parameters can be found in the Excel Report.

Design>PSC Design>PSC Design Calculation...



38	■ **Positive Moment**
39	1 Check Moment Resistance, $M_{R,d}$
40	- Design Load
41	Load Combination Name : cLCB7
42	Design Situations : Persistent & Transient
43	Load Combination Type : FX-MIN
44	M_{Ed} = 49.835 kN · m
45	
46	- factor λ, and factor η
47	λ = 0.800 (f_{ck} ≤ 50 MPa)
48	η = 1.000 (f_{ck} ≤ 50 MPa)
49	
50	- Design strength of concrete
51	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ = 22.667 MPa
52	
53	- Design strength of Reinforcement rebar
54	$f_{yd} = f_{yk} / \gamma_{s\_rebar}$ = 347.826 MPa
55	
56	- Calculate Neutral Axis
57	1) Assume neutral axis depth.
58	2) Calculate the strain of steel and tendon.
59	3) Calculate the stress of steel and tendon.
60	4) Calculate the axial force in concrete, steel, and tendon.
61	5) Check if the resultant force of cross-section is zero.
62	6) Repeat step 1 through 5 until the resultant force becomes zero.
63	
64	- Calculate F_c, F_s, F_p
65	x = 500.000 mm
66	$F_c = (\lambda \cdot x) \cdot (\eta \cdot f_{cd})$ = 8522.666 kN
67	$F_{s'} = f_{s'} \cdot A_{s'}$ = 2643.652 kN
68	$F_s = f_s \cdot A_s$ = 881.217 kN
69	$F_p = \Sigma F_{pi}$ = 17341.849 kN
70	where, x is neutral axis depth
71	
72	- Calculate moment resistance, M_{Rd}
73	$M_{Rd} = F_c \cdot a_c + F_s \cdot a_s + \Sigma (F_{pi} \cdot a_{pi})$ = 9483.236 kN · m
74	≥ M_{Ed} = 49.835 kN · m ...... OK
75	where, a_c, a_s, a_p is the distance from neutral axis depth, x to concrete, reinforcement rebar, tendon

[Fig. 1.15] Excel report for moment resistance

2. Shear resistance

Limit state of shear resistance should satisfy the condition, $V_{Ed} \le V_{Rd}$.
Shear resistance, V_{Rd}, is calculated as follows.

2.1 Design strength of material

(1) Design compressive strength of concrete.

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \tag{1.15}$$

EN1992-1-1:2004 3.1.6(1)

Using α_{cc}=1.0 for shear regardless of input value.

(2) Design yield strength of reinforcement.

$$f_{yd} = f_{yk} / \gamma_s \tag{1.16}$$

EN1992-1-1:2004 3.2.7(1)

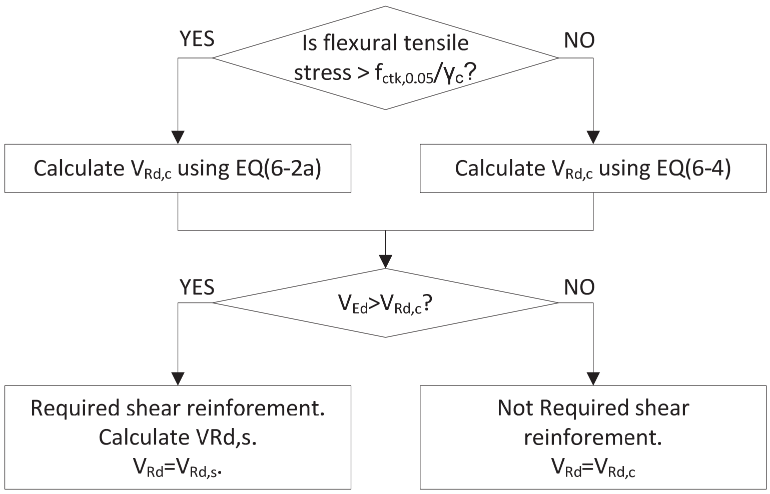
(3) Design tensile strength of tendon.

$$f_{pd} = f_{p0,1k} / \gamma_s \tag{1.17}$$

EN1992-1-1:2004 3.3.6(6)

Refer to the clause 1.1 for detail explanation of material strength.

2.2 Calculate shear resistance V_{Rd}



[Fig. 1.16] Flowchart to calculate V_{Rd}

(1) Calculate $V_{Rd,c}$

[Table 1.5] Shear strength by concrete, $V_{Rd,c}$

Flexural tensile stress	$V_{Rd,c}$
$\ge f_{ctk,0.05}/\gamma_c$	$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right]b_w d$ $V_{Rd,c} \ge (v_{min} + k_1\sigma_{cp})b_w d$
$< f_{ctk,0.05}/\gamma_c$	$V_{Rd,c} = \frac{Ib_w}{S}\sqrt{(f_{ctd})^2 + \alpha_l\sigma_{cp}f_{ctd}}$

EN1992-1-1:2004 (6.2.a), (6.2.b)

EN1992-1-1:2004 (6.4)

where,

$V_{Rd,c}$: *The design shear resistance without shear reinforcement.*

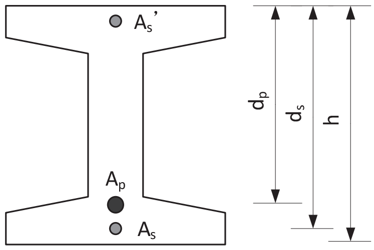
b_w : *The smallest width of the cross-section in the tensile area. When the user specifies the web thickness directly in PSC tap of Section Data dialog box, the minimum value among the specified values will be applied. When "Auto" option is selected in "Web Thick." Field, the program can automatically calculate the section size and apply the minimum value.*

d : *The effective depth of cross-section. In midas Civil, the value of "d" is calculated as the maximum value of [d_p, d_s, 0.85h].*

d_p : *Distance from the centroid of tendon to the extreme fiber of cross-section*

d_s : *Distance from the centroid of tensile rebar to the extreme fiber of cross-section*

h : *Height of section.*



[Fig. 1.17] Parameters to calculate d

$f_{ctk,0.05}$: *The characteristic axial tensile strength of concrete (5% fractile).*

$$f_{ctk,0.05} = 0.7 f_{ctm} \tag{1.18}$$

EN1992-1-1:2004 Table 3.1

f_{ctm} : *The mean value of axial tensile strength of concrete*

[Table 1.6] Mean value of axial tensile strength of concrete, f_{ctm}

Condition	f_{ctm}
≤ C50/60	$0.3f_{ck}^{2/3}$
> C50/60	$2.12 \ln(1+(f_{cm}/10))$, $f_{cm}=f_{ck}+8MPa$

EN1992-1-1:2004 Table 3.1

$$C_{Rd,c} = \frac{0.18}{\gamma_c} \tag{1.19}$$

EN1992-1-1:2004 6.2.2(1) (6.3N)

$$k = 1 + \sqrt{200/d} \leq 2.0 \tag{1.20}$$

$$\rho_l = \frac{A_{sl}}{b_w d} \leq 0.02 \tag{1.21}$$

$$k_1 = 0.15 \tag{1.22}$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2 f_{cd} \tag{1.23}$$

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2} \tag{1.24}$$

$$f_{ctd} = \frac{\alpha_{ct} f_{ck}}{\gamma_c} \tag{1.25}$$

※In midas Civil, the value of "α_l" is applied as "1.0" regardless of the tendon type.

(2) Calculate $V_{Rd,s}$

Shear resistance of members with shear reinforcement can be calculated depending on the type of shear reinforcement.

EN1992-1-1:2004 (6.8), (6.13) (6.9), (6.14) (6.12), (6.15)

[Table 1.7] $V_{Rd,s}$ and $V_{Rd,max}$, $A_{sw,max}$

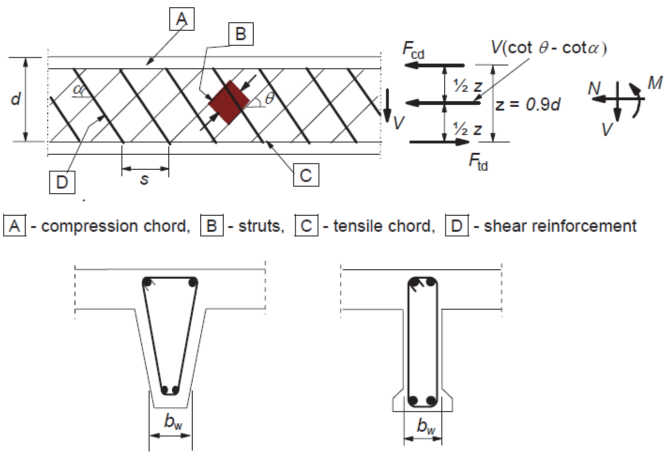
Type	Vertical shear reinforcement	Inclined shear reinforcement
$V_{Rd,s}$	$\dfrac{A_{sw}}{s} z f_{ywd} \cot\theta$	$\dfrac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha$
$V_{Rd,max}$	$\dfrac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot\theta + \tan\theta}$	$\dfrac{\alpha_{cw} b_w z v_1 f_{cd}}{1+\cot^2\theta}(\cot\theta + \cot\alpha)$
$A_{sw,max}$	$\dfrac{A_{sw,max} f_{ywd}}{b_w s} \le \dfrac{1}{2}\alpha_{cw} v_1 f_{cd}$	$\dfrac{A_{sw,max} f_{ywd}}{b_w s} \le \dfrac{1}{2}\dfrac{\alpha_{cw} v_1 f_{cd}}{\sin\alpha}$

where,

$V_{Rd,s}$: *The design value the shear force which can be sustained by the yielding shear reinforcement.*

Θ : *The angle between the concrete compression strut and the beam axis perpendicular to the shear force.*

α : *The angle between shear reinforcement and the beam axis perpendicular to the shear force.*



EN1992-1-1:2004 Figure 6.5

[Fig. 1.18] Truss model and notation for shear reinforced members

A_{sw} : *The cross-sectional area of the shear reinforcement.*

s : *The spacing of stirrups.*

z : *Inner lever arm, z=0.9d.*

f_{ywd} : *The design yield strength of the shear reinforcement.*

v_1 : *Strength reduction factor for concrete cracked in shear.*

[Table 1.8] Strength reduction factor for concrete cracked in shear, v_1

EN1992-1-1:2004 (6.10.aN),(6.10.bN)

National Annex	$f_{ywd} \ge 0.8 f_{ywk}$	$f_{ywd} < 0.8 f_{ywk}$	
		$f_{ck} \le 60\text{MPa}$	$f_{ck} \ge 60\text{MPa}$
Recommended	$0.6\left(1-\dfrac{f_{ck}}{250}\right)$	0.6	$0.9-\dfrac{f_{ck}}{200} > 0.5$
UK	$0.6\left(1-\dfrac{f_{ck}}{250}\right)$	$0.54(1-0.5\cos\alpha)$	$\left(0.84-\dfrac{f_{ck}}{200}\right)(1-0.5\cos\alpha) > 0.5$
Italy	$0.7\left(1-\dfrac{f_{ck}}{250}\right)$	0.7	$\dfrac{0.9-\dfrac{f_{ck}}{200}}{0.85} > 0.5$

α_{cw} : *Coefficient taking account of the state of the stress in the compression chord.*

[Table 1.9] Coefficient α_{cw}

Condition	α_{cw}
$0<\sigma_{cp} \le 0.25f_{cd}$	$1+\sigma_{cp}/f_{cd}$
$0.25\ f_{cd} <\sigma_{cp} \le 0.5f_{cd}$	1.25
$0.5\ f_{cd} <\sigma_{cp} \le 1.0f_{cd}$	$2.5(1-\sigma_{cp}/f_{cd})$

EN1992-1-1:2004
(6.11.aN)~(6.11.cN)

σ_{cp} : The mean compressive stress, measured positive, in the concrete due to the design axial force.

(3) Calculate shear resistance V_{Rd}.

• The shear resistance of a member with shear reinforcement.

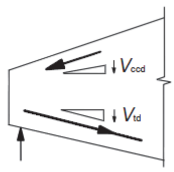
$$V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td} \tag{1.26}$$

EN1992-1-1:2004
(6.1)

where,

V_{ccd} : The design value of the shear component of the force in the compression area, in the case of an inclined compression chord.

V_{td} : The design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord.



EN1992-1-1:2004
Figure 6.2

[Fig. 1.19] Shear component for members with inclined chords

In midas civil, inclined chord is not considered. Therefore the shear resistance is calculated using shear reinforcement only.

$$V_{Rd} = V_{Rd,s}$$

• In regions of the member where $V_{Ed} \le V_{Rd,c}$ no calculate shear reinforcement is necessary.

EN1992-1-1:2004
6.2.1(3)

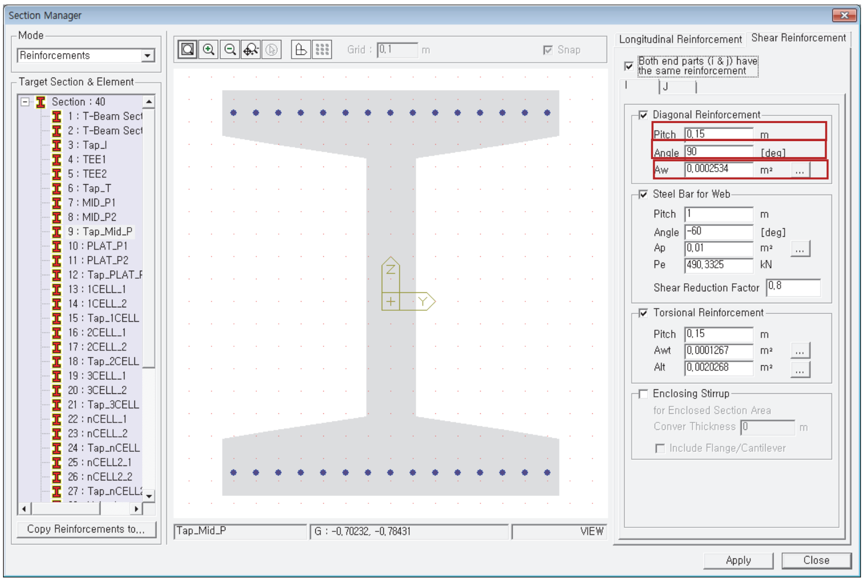
$$V_{Rd} = V_{Rd,c}$$

▭ Shear reinforcement

In midas Civil, shear reinforcement information can be defined in Section Manager.

When the shear rebar angle is entered as “90” degree, it is considered as vertical shear reinforcement. For the angle other than 90 degree, it is considered as inclined shear reinforcement.

Model>Properties>Section Manager…



s

α

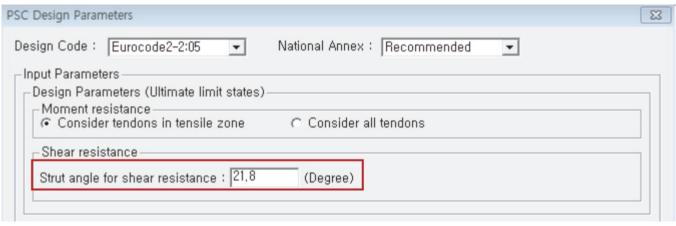
A_{sw}

[Fig. 1.20] Input shear reinforcement

Strut angle for shear resistance, θ

The angle between the concrete compression strut and the beam axis perpendicular to the shear force can be entered in PSC Design Parameters dialog box.

Design>PSC Design>PSC Design Parameters…



θ

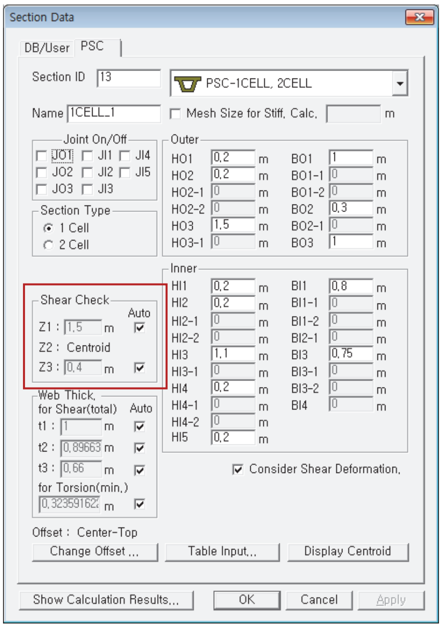
[Fig. 1.21] Input strut angle for shear resistance, θ

Smallest width of the cross-section, b_w

Shear forces are calculated at the parts critical to shear in the PSC section.

The user can directly enter the position. If the Auto option is checked on, the program checks shear at the top and bottom ends of the webs (Z1 and Z3 in the PSC Viewer dialog).

Model>Properties>Section...>PSC tab



[Fig. 1.22] Input b_w for shear resistance

2.3 Check shear resistance

$$V_{Ed} \le V_{Rd}$$

where,
V_{Ed} *: Design value of the applied shear force.*
V_{Rd} *: Design shear resistance.*

• Design load combination

In midas Civil, load combination to be used in PSC design can be defined in Results>Load combinations>Concrete Design tab. Shear resistance is verified with the most critical minimum and maximum design shear force among the load combinations specified as “ Strength/Stress” in Active column.

2.4 Check the ratio and spacing of shear reinforcement

When no shear reinforcement is required, minimum shear reinforcement should be provided.

$$\rho_w = \frac{A_{sw}}{s b_w \sin\alpha} \ge \rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \qquad (1.27)$$

EN1992-1-1:2004 (9.4),(9.5N)

$$s \le s_{l,\max} = 0.75d(1+\cot\alpha) \qquad (1.28)$$

EN1992-1-1:2004 (9.6N)

2.5 Verification of shear resistance

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check Shear Strength...

Model View | Check Combined Shear and Torsion Strength | **Check Shear Strength**

Elem	Part	Max/Min	LCom Name	Design Situations	Type	CHK	V_Ed (kN)	V_Rd (kN)	V_Rd,c (kN)	V_Rd,s (kN)	V_Rd,max (kN)	V_Ed/V_Rd
1	I[1]	Max	cLCB1	Persistent &	FX-MIN	OK	-293.1098	535.2654	535.2654	1651.6443	1651.6443	0.5476
1	I[1]	Min	cLCB7	Persistent &	FX-MAX	OK	-851.0322	1651.6443	532.1827	1651.6443	1651.6443	0.5153
1	J[2]	Max	cLCB1	Persistent &	FX-MIN	OK	-145.8806	571.1089	571.1089	1758.6953	1758.6953	0.2554
1	J[2]	Min	cLCB7	Persistent &	FX-MAX	OK	-703.8030	1758.6953	567.9218	1758.6953	1758.6953	0.4002
10	I[10]	Max	cLCB3	Persistent &	FX-MIN	OK	770.8107	1270.7025	1270.7025	2379.9263	4083.1376	0.6066
10	I[10]	Min	cLCB7	Persistent &	FX-MAX	OK	212.6846	1269.0556	1269.0556	2379.9263	4082.4858	0.1676
10	J[11]	Max	cLCB3	Persistent &	FX-MIN	OK	905.0585	1321.9133	1321.9133	2644.3626	4555.7988	0.6847
10	J[11]	Min	cLCB7	Persistent &	FX-MAX	OK	346.9323	1320.2437	1320.2437	2468.0717	4251.4169	0.2628

[Fig. 1.23] Result table for shear resistance

Elem : Element number

Part : Check location (I-End, J-End) of each element

Max./Min. : Maximum shear, minimum shear

LCom. Name : Load combination name.

Type : Displays the set of member forces corresponding to moving load case or settlement load case for which the maximum stresses are produced.

CHK : Shear strength check for element

V_Ed : Maximum shear force among Strength/Stress load combinations

V_Rd : Shear resistance.

V_Rd,c : Shear resistance of concrete.

V_Rd,s : Shear resistance of shear reinforcement.

V_Rd,max : Maximum V_Rd,s

V_Ed/V_Rd : The ratio of shear force to shear resistance.

By Excel Report

Detail design results including applied equations and design parameters can be found in the Excel Report.

Design>PSC Design>PSC Design Calculation...

134	3 Shear Resistance
135	■ **Maximum Shear Force**
136	1 Check shear resistance of Concrete, $V_{Rd,c}$
137	- Design Load
138	Load Combination Name : cLCB1
139	Design Situations : Persistent & Transient
140	Load Combination Type : FX-MIN
141	N_{Ed} = 8.339 kN
142	V_{Ed} = -293.110 kN
143	M_{Ed} = 0.049 kN · m
144	
145	- Design strength of concrete
146	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ = 26.667 MPa
147	
148	- Design strength of Reinforcement rebar
149	$f_{yd} = f_{yk} / \gamma_{s\_rebar}$ = 347.826 MPa
150	
151	- Design value for the shear resistance $V_{Rd,c}$
152	$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$ = 134.400 kN
153	$V_{Rd,c} = (I \cdot b_w / S) \cdot \sqrt{((f_{ctd})^2 + \alpha_l \cdot \sigma_{cp} \cdot f_{ctd})} \geq (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d$
154	= 535.265 kN ≥ V_{Ed} = -293.110 kN ∴ Shear reinforcement is not required
169	
170	2 Check shear resistance of Shear Reinforcement, $V_{Rd,s}$
171	- Design Parameters
172	α = 90.000 °
173	θ = 21.800 ° ($1 \leq \cot\theta \leq 2.5$, ∴ $21.8° \leq \theta \leq 45°$)
174	$z = 0.9 \cdot d$ = 1620.000 mm
175	A_{sw} = 253.400 mm^2
176	$\rho_{w,min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk}$ = 0.00126
182	
183	- Design value for the shear resistance $V_{Rd,s}$
184	Skip checking shear resistance, $V_{Rd,s}$ by shear reinforcement steel
185	
186	- Check ratio of shear reinforcement
187	$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin\alpha)$ = 0.008 ≥ $\rho_{w,min}$ = 0.001 ...... OK
188	
189	- Check Spacing of stirrups
190	$S_{l,max} = 0.75 \cdot d \cdot (1 + \cot\alpha)$ = 1350.000 mm
191	s = 150.000 mm ≤ $S_{l,max}$ = 1350.000 mm ...... OK
192	where, s is the spacing of the stirrups

[Fig. 1.24] Excel report for shear resistance

3. Torsion Resistance

The maximum resistance of a member subjected to torsion and shear is limited by the capacity of the concrete strut. In order not to exceed this resistance the following condition should be satisfied.

$$\frac{T_{Ed}}{T_{Rd,\max}} + \frac{V_{Ed}}{V_{Rd,\max}} \le 1.0 \qquad (1.29)$$

EN1992-1-1:2004
(6.29)

3.1 Design strength of material

(1) Design compressive strength of concrete.

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \qquad (1.30)$$

EN1992-1-1:2004
3.1.6(1)

※ Using α_{cc}=1.0 for torsion regardless of input value.

(2) Design yield strength of reinforcement.

$$f_{yd} = f_{yk} / \gamma_s \qquad (1.31)$$

EN1992-1-1:2004
3.2.7(1)

(3) Design tensile strength of tendon.

$$f_{pd} = f_{p0,1k} / \gamma_s \qquad (1.32)$$

EN1992-1-1:2004
3.3.6(6)

※ Refer to the clause 1.1 to see the detail explanation of material strength.

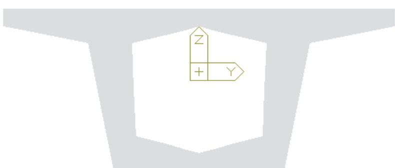
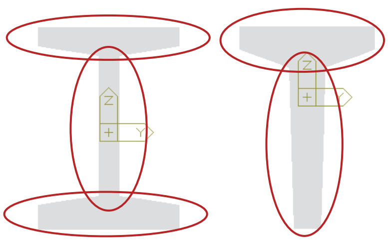
3.2 Calculate torsional resistance

(1) Check section type for torsion.

If the section is complex shapes, such as T-sections, it may be divided into a series of sub-sections.

EN1992-1-1:2004
6.3.1(3)

[Table 1.10] Section type for torsion

Closed Section Type	Open Section Type
	 sub-sections.

(2) Calculate the torsional moments over the sub-sections. (Only "Open" section type)

$$T_{Ed,i} = \frac{I_{xx,i}}{I_{xx}} T_{Ed} \qquad (1.33)$$

EN1992-1-1:2004
6.3.1(3), (5)

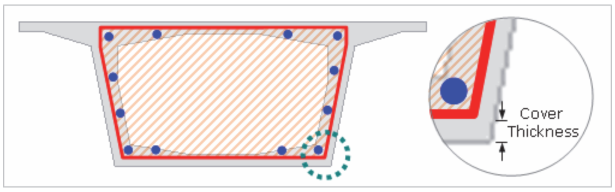
where,
$T_{Ed,i}$: The torsional moments of sub-section.
I_{xx} : The uncracked torsional stiffness of whole section.
$I_{xx,i}$: The uncracked torsional stiffness of sub-section.

(3) Calculate the transverse reinforcement required.

$$\frac{A_{st,req}}{s_t} = \frac{T_{Ed}}{2A_k f_{yd} \cot\theta} \tag{1.34}$$

EN1992-1-1:2004 (6.8),(6.26), (6.28)

where,
A_{sl} : The cross sectional area of longitudinal reinforcement.
s_t : The spacing of transverse reinforcement for torsion.
A_k : The area enclosed by the centre-lines of the connecting walls, including inner hollow areas.
u_k : The perimeter of the area A_k.



[Fig. 1.24] A_k, u_k in closed section

(4) Calculate the longitudinal reinforcement required.

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k}\cot\theta \rightarrow A_{sl,req} = \frac{T_{Ed} u_k}{2A_k f_{yd}}\cot\theta \tag{1.35}$$

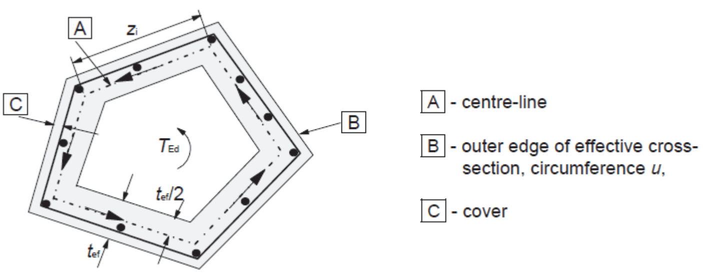
EN1992-1-1:2004 (6.28)

where,
A_{sl} : The cross sectional area of longitudinal reinforcement.
u_k : The perimeter of the area A_k.
A_k : The area enclosed by the centre-lines of the connecting walls, including inner hollow areas.

(5) Calculate design torsional resistance moment.

$$T_{Rd,\max} = 2\nu\alpha_{cw} f_{cd} A_k t_{ef,i} \sin\theta\cos\theta \tag{1.36}$$

EN1992-1-1:2004 (6.30)



[Fig. 1.25] Notations and definition for torsion

where,
ν : Strength reduction factor for concrete cracked in shear.

[Table 1.11] Strength reduction factor for concrete cracked in shear, v

National Annex	v
Recommended	$0.6\left(1-\frac{f_{ck}}{250}\right)$
UK	$0.6\left(1-\frac{f_{ck}}{250}\right)$
Italy	$0.7\left(1-\frac{f_{ck}}{250}\right)$

EN1992-1-1:2004 6.2.2(6), (6.6N)

α_{cw} : Coefficient taking account of the state of the stress in the compression chord.

[Table 1.12] Coefficient α_{cw}

Condition	α_{cw}
$0<\sigma_{cp} \le 0.25f_{cd}$	$1+\sigma_{cp}/f_{cd}$
$0.25\ f_{cd} <\sigma_{cp} \le 0.5f_{cd}$	1.25
$0.5\ f_{cd} <\sigma_{cp} \le 1.0f_{cd}$	$2.5(1-\sigma_{cp}/f_{cd})$

EN1992-1-1:2004 (6.11.aN)~(6.11.cN)

A_k : The area enclosed by the centre-lines of the connecting walls, including inner hollow areas.
$t_{ef,i}$: The effective wall thickness

$$t_{ef,i} = \frac{A}{u} \tag{1.37}$$

EN1992-1-1:2004 6.3.2(1)

A : The total area of the section within the outer circumference, including inner hollow areas.
u : The outer circumference of the section.

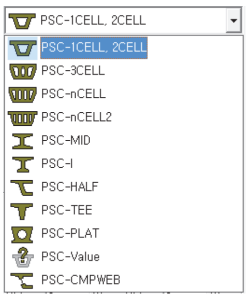
▭ Section Type for torsion

In midas Civil, closed type and number of division for PSC DB sections are shown in the table below. Closed type section has zero number of divisions since it is considered as a unified section.

[Table 1.13] Section type and sub-sections for torsion

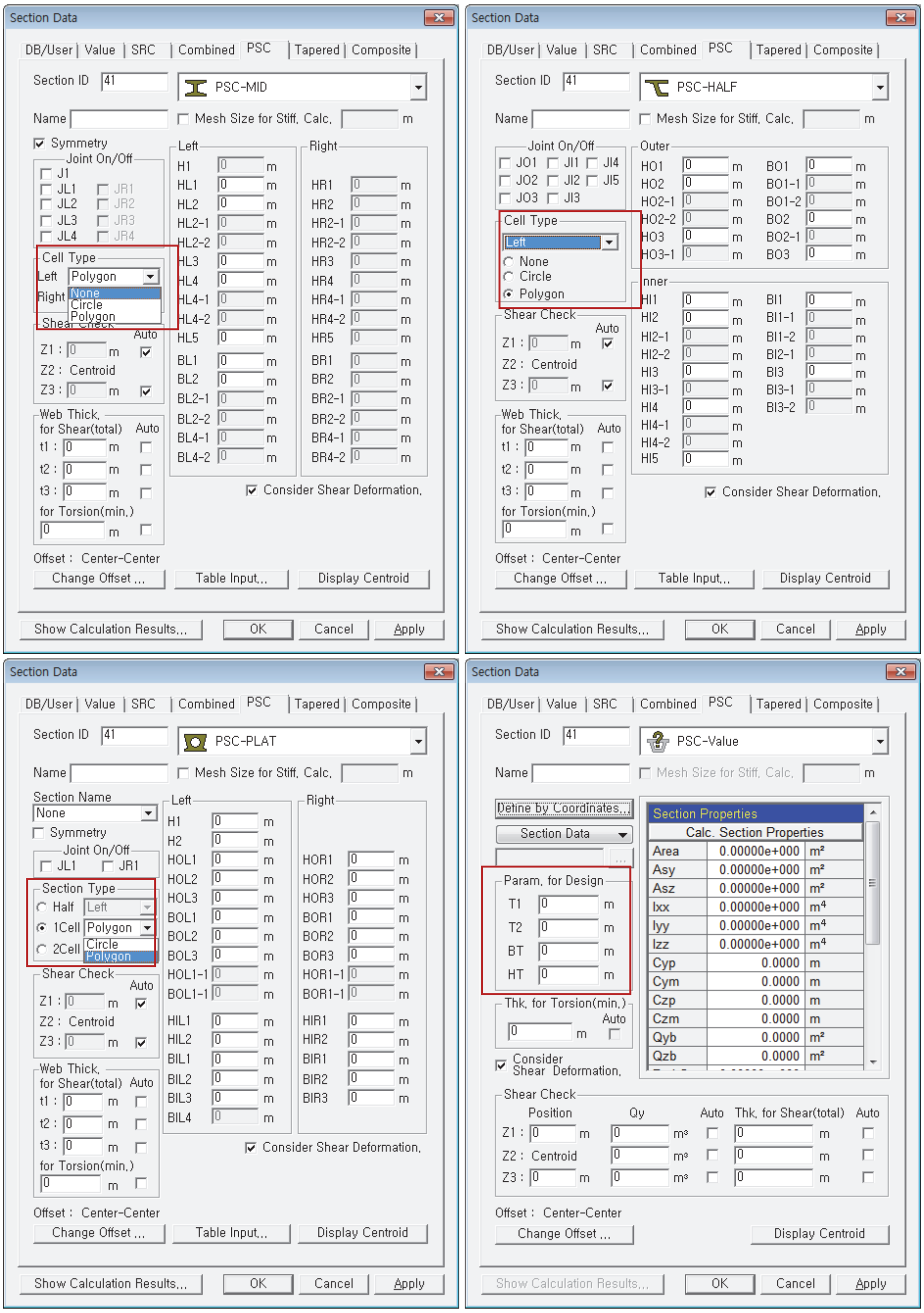
Section Shape		Closed Type	No of divisions
PSC-1CELL	-	Closed	-
PSC-2CELL	-	Closed	-
PSC-3CELL	-	Closed	-
PSC-nCELL	-	Closed	
PSC-nCELL2	-	Closed	-
PSC-MID	None	Open	0
	Circle	Open	3
	Polygon	Open	3
PSC-I	-	Open	3
PSC-HALF	None	Open	2
	Circle	Open	3
	Polygon	Open	3
PSC-TEE	-	Open	2
PSC-PLAT	Half	Open	3
	1Cell	Closed	-
	2Cell	Closed	-
PSC-VALUE	T1>0 and HT>0	Closed	-
	T1>0 and T2>0	Open	3
	T1>0 and T2=0, T1=0 and T2>0	Open	2
	T1=0 and T2=0	Open	0
PSC-CMPWEB	-	Closed	-

• PSC Section Type



[Fig. 1.26] PSC section type list in program

• Sub type of each section type.



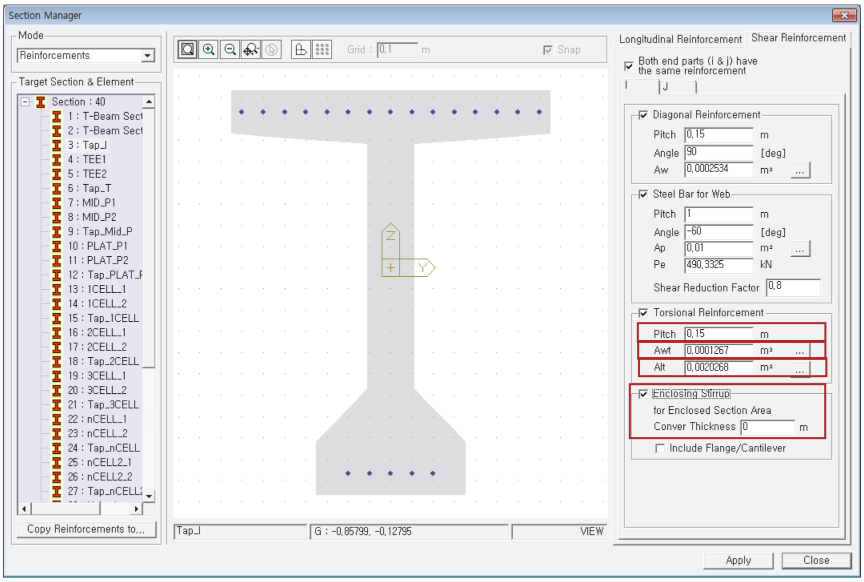
[Fig. 1.28] Sub type of section

☐ Parameters for torsion

In midas Civil, when calculating A_k and u_k, section area and perimeter of closed section are calculated based on the cover thickness entered in Section Manager. In order to calculate them based on the center line as specified in Eurocode, enter the cover thickness value as “section thickness * ½”.

Transverse and longitudinal reinforcement for torsion can also be defined.

Model>Properties>Section Manager...>Reinforcements



s_t
A_{st}
A_{sl}
Cover thickness for A_k, u_k.
(at Center line)

[Fig. 1.29] Section Manager Dialog

3.3 Check torsional moment resistance

$$T_{Ed} \le T_{Rd,\max} \tag{1.38}$$

EN1992-1-1:2004 (6.29)

$$\frac{T_{Ed}}{T_{Rd,\max}} + \frac{V_{Ed}}{V_{Rd,\max}} \le 1.0 \tag{1.39}$$

where,

T_{Ed} : *The design torsional moment.*
V_{Ed} : *The design transverse force.*
$T_{Rd,max}$: *The design torsional resistance moment.*
$V_{Rd,max}$: *The maximum design shear resistance.*

• Design load combination

In midas Civil, load combination to be used in PSC design can be defined in Results>Load combinations>Concrete Design tab. Torsional resistance is verified with the most critical minimum and maximum design torsional force among the load combinations specified as " Strength/Stress" in Active column.

3.4 Check reinforcement

$$A_{sl,req} \le A_{sl}$$

$$s_t = \min\left[u/8, s_{l,\max}\right] \tag{1.40}$$

EN1992-1-1:2004 9.2.3(3)

where,

A_{sl} : *The cross sectional area of longitudinal reinforcement.*
u : *The outer circumference of the section.*
s_t : *The longitudinal spacing.*
$s_{l,max}$: *The maximum longitudinal spacing between shear assemblies.*

$$s_{l,\max} = 0.75d\left(1+\cot\alpha\right) \tag{1.41}$$

3.5 Verify torsional resistance

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check Combined Shear and Torsion Strength…

Model View | Check Combined Shear and Torsion Strength

Elem	Part	Max/Min	LCom Name	Design Situations	Type	CHK	T_Ed (kN·m)	T_Rd,max (kN·m)	V_Ed (kN)	V_Rd,max (kN)	Ratio
1	I[1]	T-Max	cLCB1	Persistent &	MZ-MIN	OK	-200.0295	905.7455	-664.6445	1651.6443	0.6052
1	I[1]	V-Max	cLCB1	Persistent &	MZ-MAX	OK	-199.5906	905.7455	-293.1098	1651.6443	0.3798
1	I[1]	V-Min	cLCB7	Persistent &	MZ-MIN	OK	-108.3941	905.7455	-851.0322	1651.6443	0.6252
1	J[2]	T-Max	cLCB1	Persistent &	MZ-MIN	OK	-200.0295	931.2917	-517.4153	1758.6953	0.4918
1	J[2]	V-Max	cLCB1	Persistent &	MZ-MAX	OK	-199.5906	931.2917	-145.8806	1758.6953	0.2801
1	J[2]	V-Min	cLCB7	Persistent &	MZ-MIN	OK	-108.3941	931.2917	-703.8030	1758.6953	0.5073
10	I[10]	T-Max	cLCB3	Persistent &	MZ-MAX	OK	73.7545	9709.2704	399.1352	4082.6981	0.1054
10	I[10]	V-Max	cLCB3	Persistent &	MZ-MIN	OK	71.5761	9710.3156	770.8107	4083.1376	0.1962
10	I[10]	V-Min	cLCB7	Persistent &	MZ-MAX	OK	31.3918	9708.7656	212.6846	4082.4858	0.0553
10	J[11]	T-Max	cLCB3	Persistent &	MZ-MAX	OK	73.7545	10202.6439	533.3830	4251.6325	0.1327
10	J[11]	V-Max	cLCB3	Persistent &	MZ-MIN	OK	71.5761	10203.7150	905.0585	4555.7988	0.2057
10	J[11]	V-Min	cLCB7	Persistent &	MZ-MAX	OK	31.3918	10202.1266	346.9323	4251.4169	0.0847

[Fig. 1.30] Result table for torsion resistance

Elem : Element number

Part : Check location (I-End, J-End) of each element

Max./Min.: Maximum torsion/shear, minimum torsion/shear

LCom Name: Load combination name.

Type: Displays the set of member forces corresponding to moving load case or settlement load case for which the maximum stresses are produced.

CHK: Shear and torsion strength check for element

T_Ed: Maximum torsional moment among Strength/Stress load combinations

T_Rd,max: Design torsional resistance moment.

V_Ed: Maximum shear force among Strength/Stress load combinations

V_Rd,max: The maximum shear resistance of the section.

Ratio: The ratio $T_{Ed}/T_{Rd,\,max} + V_{Ed}/_{VRd,max}$

By Excel Report

Detail design results including applied equations and design parameters can be found in the Excel Report.

Design>PSC Design>PSC Design Calculation…

292	4 Torsional Resistance
293	■ **Maximum Shear Force**
294	1 Design Load
295	Load Combination Name : cLCB1
296	Design Situations : Persistent & Transient
297	Load Combination Type : MZ-MAX
298	N_{Ed} = 8.339 kN
299	V_{Ed} = -293.110 kN
300	T_{Ed} = -199.591 kN · m
301	M_{Ed} = 0.049 kN · m
302	
303	- Design strength of concrete
304	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c$ = 26.667 MPa
305	
306	- Design strength of Reinforcement rebar
307	$f_{yd} = f_{yk} / \gamma_{s\_rebar}$ = 347.826 MPa
308	
309	2 Check Torsional Resistance
310	■ Top Flange
311	- Design Parameters
312	$t_{ef,i} = A_{ki} / u_i$ = 100.767 mm
313	$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta)$ = 0.076 mm^2/mm
314	$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd})$ = 0.474 mm^2/mm
315	where, $T_{Ed,i} = T_{Ed} \cdot I_{xx\_i} / I_{xx}$ = 45.218 kN · m
316	I_{xx_i} = 5.6992E+09 mm^4
317	I_{xx} = 2.5156E+10 mm^4
318	u_i = 3403.900 mm
319	A_{ki} = 343000.000 mm^2
320	$t_{ef,i}$ is the effective wall thickness
321	u_i is the outer circumference of the cross-section
322	A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas
323	
324	
325	- Calculate maximum torsion resistance, $T_{Rd,max}$
326	$T_{Rd,max} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_{ef,i} \cdot \sin\theta \cdot \cos\theta$ = 320.347 kN · m $\geq T_{Ed,i}$ ...... OK
327	where, $v = 0.6 \cdot (1 - f_{ck} / 250)$ = 0.504
328	α_{cw} = 1.0 = 1.000 (for non-prestressed structures)
329	$\sigma_{cp} = N_{Ed} / A_c$ = -0.009 MPa
330	N_{Ed} = -8.339 kN (if compression, N_{Ed} > 0)
331	A_c = 886000.000 mm^2

[Fig. 1.31] Excel report for torsion resistance

Serviceability Limit States

1. Stress for cross section at a construction stage

For stress verification in the construction stage, the following condition should be satisfied.

The most critical compressive stress during the construction stage ≤ Allowable compressive stress of concrete before losses : $\sigma_c \leq \sigma_{ca}$

The most critical tensile stress during the construction stage ≤ Allowable tensile stress of concrete before losses : $\sigma_t \leq \sigma_{ta}$

1.1 Allowable stress of concrete

(1) Allowable compressive stress of concrete

[Table 1.14] Allowable compressive stress of concrete, σ_{ca}

Tendon Type	σ_{ca}
Post-tension	$k_1 f_{ck}(t)$
Pre-tension	$k_6 f_{ck}(t)$

EN1992-1-1:2004
3.1.2(5)

where,

k_1, k_6 : If "User Input Data" option is checked on, the coefficients of "k_1" and "k_6" will be applied as the user defined value. However, if the option is checked off, the values in Table 1.15 will be applied.

[Table 1.15] Coefficient k_1, k_6

National Annex	k_1	k_6
Recommended	0.6	0.7
UK	0.6	0.7
Italy	0.65	0.65

t : The age of concrete in days.

$f_{ck}(t)$: The concrete compressive strength at time t for a number of stages.

[Table 1.16] Concrete compressive strength at t, $f_{ck}(t)$

Condition	$f_{ck}(t)$
3 < t < 28 days	$f_{cm}(t)$ – 8MPa
t ≥ 28days	f_{ck}

EN1992-1-1:2004
Table 3.1

$f_{cm}(t)$: The mean concrete compressive strength of concrete at an age of t days.

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \tag{1.43}$$

EN1992-1-1:2004
(3.1)

f_{cm} : The mean compressive strength at 28 days.

$$f_{cm} = f_{ck} + 8MPa \tag{1.44}$$

EN1992-1-1:2004)
Table 3.1

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t} \right)^{1/2} \right] \right\} \quad (1.45)$$

EN1992-1-1:2004 (3.2)

s : A coefficient which depends on the type of cement.

[Table 1.17] Coefficient s

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Cement Class	s
Class R	0.20
Class N	0.25
Class S	0.38

(2) Allowable tensile stress of concrete

[Table 1.18] Allowable tensile stress of concrete, σ_{ta}

Tendon Type	σ_{ta}
Post-tension	$k_1 f_{ctm}(t)$
Pre-tension	$k_1 f_{ctm}(t)$

where,
$f_{ctm}(t)$: The mean concrete tensile strength of concrete at an age of t days.

$$f_{ctm}(t) = \left(\beta_{cc}(t)\right)^{\alpha} f_{ctm} \quad (1.46)$$

EN1992-1-1:2004 (3.4)

f_{ctm} : The mean value of axial tensile strength of concrete.

[Table 1.19] Mean value of axial tensile strength, f_{ctm}

EN1992-1-1:2004 Table 3.1

Condition	f_{ctm}
≤ C50/60	$0.30 f_{ck}^{2/3}$
> C50/60	$2.12 \ln(1+(f_{cm}/10))$

$\beta_{cc}(t)$: A coefficient which depends on the age of the concretet.
α : A coefficient for $f_{ctm}(t)$

[Table 1.20] Coefficient α

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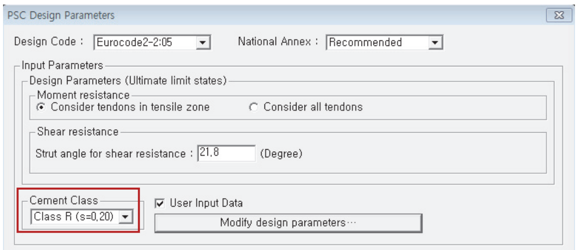
Condition	α
t < 28 days	1
t ≥ 28 days	2/3

• If both post-tension and pre-tension type tendons are assigned in a cross-section, the tendon type will be determined as the type which has larger tendon area.

Cement Class

Cement class can be defined in PSC Design Parameters dialog box.

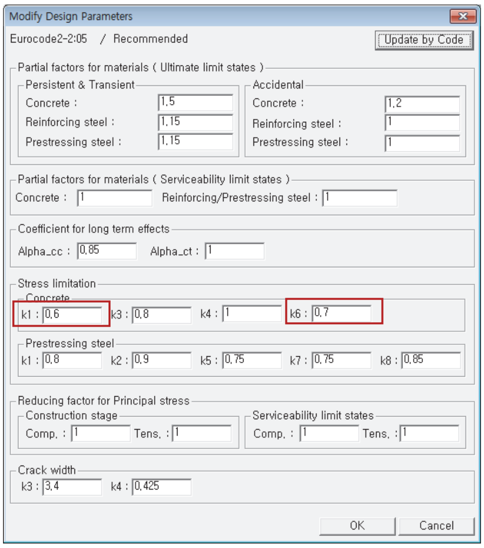
- *Design>PSC Design>PSC Design Parameters…*



[Fig. 1.32] Input Cement Class

Coefficient k_1, k_6 for Concrete

- *Design>PSC Design>PSC Design Parameters>Modify Design Parameters…*

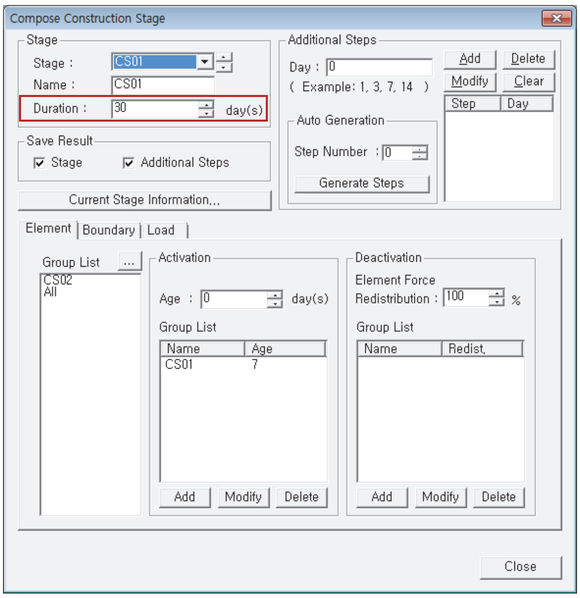


[Fig. 1.33] Input coefficient k_1, k_6 for stress limitation

Age of concrete

Age of concrete, t, will be applied as the day in the duration field in Compose Construction Stage dialog box.

- *Loads>Construction Stage Analysis Data>Define Construction Stage…>Compose Construction Stage*



[Fig. 1.34] Input age of concrete

1.2 Check stress for cross section at a construction stage

$$\sigma_c \leq \sigma_{ca}\ ,\quad \sigma_t \leq \sigma_{ta} \tag{1.47}$$

1.3 Verification of stress for cross section at a construction stage

By Result Tables

The design results can be checked in the table below.

- *Design>PSC Design>PSC Design Result Tables>Check stress for cross section* at a construction *stage…*

Model View | Check stress for cross section at a construction stage

Elem	Part	Comp./Tens.	Stage	CHK	FT (N/mm²)	FB (N/mm²)	FTL (N/mm²)	FBL (N/mm²)	FTR (N/mm²)	FBR (N/mm²)	FMAX (N/mm²)	ALW (N/mm²)
1	I[1]	Compression	CS02	NG	7.1854	0.4965	-21.5794	-12.4266	37.8267	15.2962	37.8267	24.0000
1	J[2]	Compression	CS02	OK	16.5726	8.4235	10.1651	5.5420	23.3872	11.7123	23.3872	24.0000
10	I[10]	Compression	CS04	OK	9.4801	19.4527	9.2955	19.2807	9.6715	19.6316	19.6316	24.0000
10	J[11]	Compression	CS02	OK	17.0273	12.6948	15.3387	11.1209	18.7778	14.3306	18.7778	24.0000

[Fig. 1.35] Result table for stress at a construction stage

Elem : Element number
Part : Check location (I-End, J-End) of each element
Comp./Tens.: Compression or Tension Stress
Stage : Construction stage at which stresses are maximum at the corresponding section.
CHK : Combined stress check for construction stages
FT : Combined Stress due to M_y *and axial force at Top fiber*
FB : Combined Stress due to M_y *and axial force at Bottom fiber*
FTL : Combined Stress due to M_y*,* M_z *and axial force at Top Left fiber*
FBL : Combined Stress due to M_y*,* M_z *and axial force at Bottom Left fiber*
FTR : Combined Stress due to M_y*,* M_z *and axial force at Top Right fiber*
FBR : Combined Stress due to M_y*,* M_z *and axial force at Bottom Right fiber*
FMAX : Maximum combined stress out of the above six components.
ALW : Allowable stress of cross section at construction stage.

2. Stress for cross section at service loads

Stress due to service load combinations after losses should satisfy the following conditions:

Maximum compressive stress of concrete after losses ≤ Allowable compressive stress of concrete : $\sigma_c \le \sigma_{ca}$

Maximum tensile stress of concrete after losses ≤ Allowable tensile stress of concrete : $\sigma_t \le \sigma_{ta}$

2.1 Allowable stress of concrete

(1) Allowable compressive stress of concrete

[Table 1.21] Allowable compressive stress of concrete, σ_{ca}

Tendon Type	σ_{ca}
Post-tension	$k_1 f_{ck}$
Pre-tension	$k_6 f_{ck}$

(2) Allowable tensile stress of concrete

[Table 1.22] Allowable tensile stress of concrete, σ_{ta}

Tendon Type	σ_{ta}
Post-tension	$k_1 f_{ctm}$
Pre-tension	$k_1 f_{ctm}$

※ Refer to the clause 1.1 for the detailed explanation.

2.2 Check stress for cross section at a service loads

$$\sigma_c \le \sigma_{ca}, \quad \sigma_t \le \sigma_{ta} \tag{1.48}$$

2.3 Verification of stress for cross section at a service loads

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check stress for cross section at service loads…

Model View | Check stress for cross section at service loads

	Elem	Part	Comp./Tens.	LCom Name	Type	CHK	FT (N/mm²)	FB (N/mm²)	FTL (N/mm²)	FBL (N/mm²)	FTR (N/mm²)	FBR (N/mm²)	FMAX (N/mm²)	ALW (N/mm²)
▶	1	I[1]	Compression	cLCB17	FX-MIN	NG	14.4088	0.9619	-12.9861	-11.3459	43.5907	15.0566	43.5907	24.0000
	1	J[2]	Compression	cLCB17	FX-MAX	NG	15.2667	-0.2215	-7.5950	-10.5029	39.5812	11.5127	39.5812	24.0000
	10	I[10]	Compression	cLCB17	FX-MAX	NG	17.4530	-3.2980	10.1903	-10.0674	24.9881	3.7438	24.9881	24.0000
	10	J[11]	Compression	cLCB17	FX-MAX	NG	16.6431	-2.4612	8.3472	-10.1939	25.2430	5.5755	25.2430	24.0000

[Fig. 1.36] Result table for stress at a service loads

Comp./Tens.: Compression or Tension Stress

Type: Displays the set of member forces corresponding to moving load case or settlement load case for which the maximum stresses are produced

FT: Combined Stress due to M_y and axial force at Top fiber

FB: Combined Stress due to M_y and axial force at Bottom fiber

FTL: Combined Stress due to M_y, M_z and axial force at Top Left fiber

FBL: Combined Stress due to M_y, M_z and axial force at Bottom Left fiber

FTR: Combined Stress due to M_y, M_z and axial force at Top Right fiber

FBR: Combined Stress due to M_y, M_z and axial force at Bottom Right fiber

FMAX: Maximum combined stress out of the above six components.

ALW: Allowable stress in concrete at service limit state.

3. Tensile stress for Prestressing tendons

Verify the induced stress and allowable stress of tendon by tendon groups.

Before losses, tendon stress at the anchor right after grouting ≤ Allowable stress

After immediate losses, maximum tendon stress ≤ Allowable stress

After all losses, maximum tendon stress ≤ Allowable stress

3.1 Allowable stress of tendon

(1) Allowable stress in tendon immediately after anchor set at anchorages

$$\sigma_{p,\max} = \min\left[k_1 f_{pk}, \quad k_2 f_{p0.1k}\right] \tag{1.49}$$

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where,

k_1, k_2 : If "User Input Data" option is checked on, the coefficients of "k_1" and "k_2" will be applied as the user defined value. However, if the option is checked off, the values in Table 1.23 will be applied.

[Table 1.23] Coefficient k_1, k_2

National Annex	k_1	k_2
Recommended	0.8	0.9
UK	0.8	0.9
Italy	0.8	0.9

f_{pk} : Characteristic tensile strength of prestressing steel.

$f_{p0.1k}$: Characteristic 0.1% proof-stress of prestressing steel.

(2) Allowable Stress in Tendon immediately after anchor set elsewhere

$$\sigma_{pm0}(x) = \min\left[k_7 f_{pk}, \quad k_8 f_{p0.1k}\right] \tag{1.50}$$

EN1992-1-1:2004 5.10.3(2)

where,

k_7, k_8 : If "User Input Data" option is checked on, the coefficients of "k_7" and "k_8" will be applied as the user defined value. However, if the option is checked off, the values in Table 1.24 will be applied.

[Table 1.24] Coefficient k_7, k_8

National Annex	k_7	k_8
Recommended	0.75	0.85
UK	0.75	0.85
Italy	0.75	0.85

f_{pk} : Characteristic tensile strength of prestressing steel.

$f_{p0.1k}$: Characteristic 0.1% proof-stress of prestressing steel.

(3) Allowable stress in tendon at service limit state after losses

$$\sigma_p = k_5 f_{pk} \tag{1.51}$$

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where,

k_5 : If "User Input Data" option is checked on, the coefficient of "k_5" will be applied as the user defined value. However, if the option is checked off, the value in Table 1.25 will be applied.

[Table 1.25] Coefficient k_5

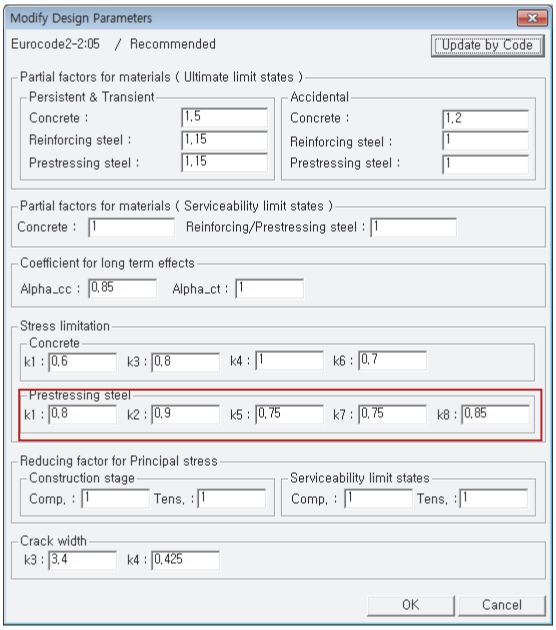
National Annex	k_5
Recommended	0.75
UK	0.75
Italy	0.6

f_{pk} : Characteristic tensile strength of prestressing steel.

Coefficient for tendons

Parameters used in calculating tendon stress can be defined in PSC Design Parameters dialog box.

Design>PSC Design>PSC Design Parameters>Modify Design Parameters…

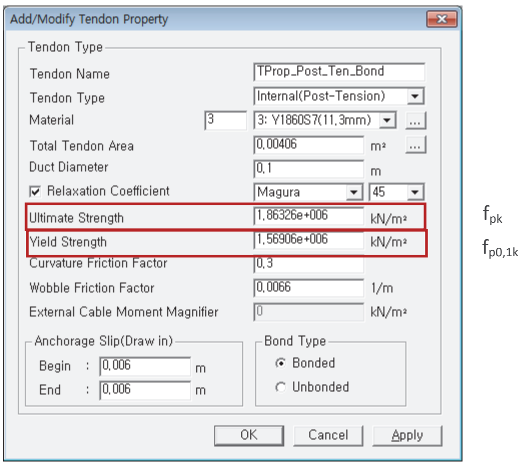


[Fig. 1.37] Input coefficient of prestressing steel in SLS

Strength of tendon

Load > Prestress Loads > Tendon Property

Tendon strength can be entered in Tendon Properties dialog box.



[Fig. 1.38] Input tendon strength, f_{pk}, $f_{p0,1k}$

3.2 Check tensile stress for prestressing tendons

(1) Post-tension tendon

- Stress in tendon at anchorages ≤ min[$k_1 f_{pk}$, $k_2 f_{p0.1k}$]
- Maximum tress in tendon along the length of the member away from anchorages ≤ min[$k_7 f_{pk}$, $k_8 f_{p0.1k}$]
- Maximum stress in tendon after all losses at the last stage ≤ $k_5 f_{pk}$

(2) Pre-tension tendon

- Stress in tendon ≤ min[$k_1 f_{pk}$, $k_2 f_{p0.1k}$]
- Stress in tendon after all losses at the last stage ≤ $k_5 f_{pk}$

3.3 Verification of stress for cross section at a service loads

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check tensile stress for prestressing tendons …

Model View | Check tensile stress for Prestressing tendons

Tendon	FDL1 (N/mm²)	FDL2 (N/mm²)	FLL1 (N/mm²)	AFDL1 (N/mm²)	AFDL2 (N/mm²)	AFLL1 (N/mm²)
N1-1	1189.1527	1285.6753	1175.5123	1255.2512	1176.7980	1176.7980

[Fig. 1.39] Result table for tensile stress for prestressing tendons

Tendon: Tendon profile name.

For Post-tensioned:

FDL1: Stress in tendon at anchorages.

FDL2: Maximum stress in tendon along the length of the member away from anchorages, immediately after anchor set.

FLL1: Maximum stress in tendon after all losses at the last stage.

AFDL1: Allowable stress in tendon immediately after anchor set at anchorages.

AFDL2: Allowable stress in tendon immediately after anchor set elsewhere.

AFLL1: Allowable stress in tendon at service limit state after losses.

For Pre-tensioned:

FDL1: Stress in tendon.

FDL2: -

FLL1: Maximum stress in tendon after all losses at the last stage.

AFDL1: Allowable stress in tendon prior to transfer.

AFDL2: -

AFLL1: Allowable stress in tendon at service limit state after losses.

4. Principal stress at a construction stage

Verify the principal stress during the construction stage at the stress verification point 1~10 defined in Section Manager dialog box.

Maximum principal stress during the construction stage ≤ Allowable stress

4.1 Allowable tensile stress

$$\sigma_{ca} = k_t f_{ctm}(t) \tag{1.52}$$

where,

k_t = If "User Input Data" option is checked on, the coefficient of "k_t" will be applied as the user defined value. However, if the option is checked off, "0.6" will be applied.

$f_{ctm}(t)$: The mean compressive strength at an age of t days. Refer to the clause 1.1 for the calculation of "$f_{ctm}(t)$".

4.2 Maximum principal stress

Maximum principal stress during the construction stage can be calculated as the following equation.

$$\sigma_{ps} = \frac{1}{2}\left[\left(\sigma_x + \sigma_z\right) \pm \sqrt{\left(\sigma_x - \sigma_z\right)^2 + 4\left(\tau_s + \tau_t + \tau_p\right)^2}\right] \tag{1.53}$$

where,

σ_x : Sum of axial stresses in ECS x-direction

σ_z : Sum of axial stresses in ECS z-direction

τ_s : Shear stress due to shear.

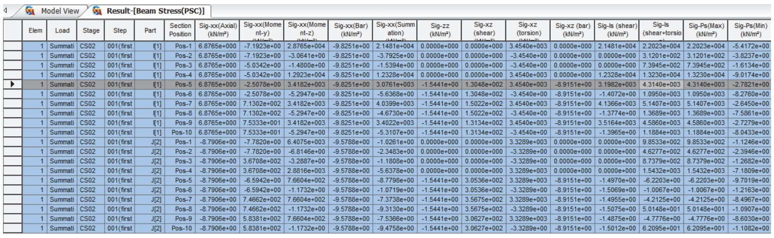
τ_t : Shear stress due to torsion.

τ_p : Shear stress due to shear reinforcement.

Beam stresses of PSC

Stress component to calculate the maximum principal stress can be checked in the table below.

Results>Result Tables>Beam>Stress(PSC)...


Model View | Result-[Beam Stress(PSC)]

Elem	Load	Stage	Step	Part	Section Position	Sig-xx(Axial) (kN/m²)	Sig-xx(Moment-y)	Sig-xx(Moment-z)	Sig-xx(Bar) (kN/m²)	Sig-xx(Summation)	Sig-zz (kN/m²)	Sig-xz (shear)	Sig-xz (torsion)	Sig-xz (bar) (kN/m²)	Sig-ls (shear) (kN/m²)	Sig-ls (shear+torsio	Sig-Ps(Max) (kN/m²)	Sig-Ps(Min) (kN/m²)
1	Summati	CS02	001(first	I[1]	Pos-1	6.8765e+000	-7.1923e+00	2.8765e+004	-9.8251e+00	2.1481e+004	0.0000e+000	0.0000e+000	3.4540e+003	0.0000e+000	2.1481e+004	2.2023e+004	2.2023e+004	-5.4172e+00
1	Summati	CS02	001(first	I[1]	Pos-2	6.8765e+000	-7.1923e+00	-3.0641e+00	-9.8251e+00	-3.7925e+00	0.0000e+000	0.0000e+000	-3.4540e+00	0.0000e+000	0.0000e+000	3.1201e+002	3.1201e+002	-3.8237e+00
1	Summati	CS02	001(first	I[1]	Pos-3	6.8765e+000	-5.0342e+00	-1.4800e+00	-9.8251e+00	-1.5394e+00	0.0000e+000	0.0000e+000	-3.4540e+00	0.0000e+000	0.0000e+000	7.3945e+002	7.3945e+002	-1.6134e+00
1	Summati	CS02	001(first	I[1]	Pos-4	6.8765e+000	-5.0342e+00	1.2923e+004	-9.8251e+00	1.2328e+004	0.0000e+000	0.0000e+000	3.4540e+003	0.0000e+000	1.2328e+004	1.3230e+004	1.3230e+004	-9.0174e+00
1	Summati	CS02	001(first	I[1]	Pos-5	6.8765e+000	-2.5078e+00	3.4182e+003	-9.8251e+00	3.0761e+003	-1.5441e+00	1.3048e+002	3.4540e+003	-8.9151e+00	3.1982e+003	4.3140e+003	4.3140e+003	-2.7821e+00
1	Summati	CS02	001(first	I[1]	Pos-6	6.8765e+000	-2.5078e+00	-5.2947e+00	-9.8251e+00	-5.6368e+00	-1.5441e+00	1.3048e+002	-3.4540e+00	-8.9151e+00	-1.4072e+00	1.0950e+003	1.0950e+003	-8.2760e+00
1	Summati	CS02	001(first	I[1]	Pos-7	6.8765e+000	7.1302e+002	3.4182e+003	-9.8251e+00	4.0399e+003	-1.5441e+00	1.5022e+002	3.4540e+003	-8.9151e+00	4.1366e+003	5.1407e+003	5.1407e+003	-2.6450e+00
1	Summati	CS02	001(first	I[1]	Pos-8	6.8765e+000	7.1302e+002	-5.2947e+00	-9.8251e+00	-4.6730e+00	-1.5441e+00	1.5022e+002	-3.4540e+00	-8.9151e+00	-1.3774e+00	1.3689e+003	1.3689e+003	-7.5861e+00
1	Summati	CS02	001(first	I[1]	Pos-9	6.8765e+000	7.5333e+001	3.4182e+003	-9.8251e+00	3.4022e+003	-1.5441e+00	1.3134e+002	3.4540e+003	-8.9151e+00	3.5164e+003	4.5860e+003	4.5860e+003	-2.7279e+00
1	Summati	CS02	001(first	I[1]	Pos-10	6.8765e+000	7.5333e+001	-5.2947e+00	-9.8251e+00	-5.3107e+00	-1.5441e+00	1.3134e+002	-3.4540e+00	-8.9151e+00	-1.3965e+00	1.1884e+003	1.1884e+003	-8.0433e+00
1	Summati	CS02	001(first	J[2]	Pos-1	-8.7906e+00	-7.7820e+00	6.4075e+003	-9.5788e+00	-1.0261e+00	0.0000e+000	0.0000e+000	3.3289e+003	0.0000e+000	0.0000e+000	9.8533e+002	9.8533e+002	-1.1246e+00
1	Summati	CS02	001(first	J[2]	Pos-2	-8.7906e+00	-7.7820e+00	-6.8146e+00	-9.5788e+00	-2.3483e+00	0.0000e+000	0.0000e+000	-3.3289e+00	0.0000e+000	0.0000e+000	4.6277e+002	4.6277e+002	-2.3946e+00
1	Summati	CS02	001(first	J[2]	Pos-3	-8.7906e+00	3.6708e+002	-3.2887e+00	-9.5788e+00	-1.1808e+00	0.0000e+000	0.0000e+000	-3.3289e+00	0.0000e+000	0.0000e+000	8.7379e+002	8.7379e+002	-1.2682e+00
1	Summati	CS02	001(first	J[2]	Pos-4	-8.7906e+00	3.6708e+002	2.8816e+003	-9.5788e+00	-5.6378e+00	0.0000e+000	0.0000e+000	3.3289e+003	0.0000e+000	0.0000e+000	1.5432e+003	1.5432e+003	-7.1809e+00
1	Summati	CS02	001(first	J[2]	Pos-5	-8.7906e+00	-6.5942e+00	7.6604e+002	-9.5788e+00	-8.7798e+00	-1.5441e+00	3.0536e+002	3.3289e+003	-8.9151e+00	-1.4970e+00	-6.2203e+00	-6.2203e+00	-9.7019e+00
1	Summati	CS02	001(first	J[2]	Pos-6	-8.7906e+00	-6.5942e+00	-1.1732e+00	-9.5788e+00	-1.0719e+00	-1.5441e+00	3.0536e+002	-3.3289e+00	-8.9151e+00	-1.5069e+00	-1.0067e+00	-1.0067e+00	-1.2163e+00
1	Summati	CS02	001(first	J[2]	Pos-7	-8.7906e+00	7.4662e+002	7.6604e+002	-9.5788e+00	-7.3738e+00	-1.5441e+00	3.5675e+002	3.3289e+003	-8.9151e+00	-1.4955e+00	-4.2125e+00	-4.2125e+00	-8.4967e+00
1	Summati	CS02	001(first	J[2]	Pos-8	-8.7906e+00	7.4662e+002	-1.1732e+00	-9.5788e+00	-9.3130e+00	-1.5441e+00	3.5675e+002	-3.3289e+00	-8.9151e+00	-1.5075e+00	5.0148e+001	5.0148e+001	-1.0907e+00
1	Summati	CS02	001(first	J[2]	Pos-9	-8.7906e+00	5.8381e+002	7.6604e+002	-9.5788e+00	-7.5366e+00	-1.5441e+00	3.0627e+002	3.3289e+003	-8.9151e+00	-1.4875e+00	-4.7776e+00	-4.7776e+00	-8.6030e+00
1	Summati	CS02	001(first	J[2]	Pos-10	-8.7906e+00	5.8381e+002	-1.1732e+00	-9.5788e+00	-9.4758e+00	-1.5441e+00	3.0627e+002	-3.3289e+00	-8.9151e+00	-1.5012e+00	6.2095e+001	6.2095e+001	-1.1082e+00

[Fig. 1.40] Result table for beam stress

Sig-xx (Axial): Axial stress due to the axial force (Fx) in the ECS x-direction

Sig-xx (Moment-y): Stress due to My (moment about the ECS y-axis) in ECS x-direction

Sig-xx (Moment-z): Stress due to Mz (moment about the ECS z-axis) in ECS x-direction

Sig-xx (Bar): Axial stress due to shear steel bars in the ECS x-direction

Sig-xx (Summation): Sum of the axial stress in the ECS x-direction and the axial stress due to shear steel bars in the ECS x-direction

Sig-zz: Stress in the ECS z-direction

Sig-xz (shear): Sum of shear stresses due to shear force and shear steel bars

Sig-xz (torsion): Shear stress due to torsion

Sig-xz (bar): Shear stress due to shear steel bars

Sig-Is (shear): Diagonal stress due to shear force
Sig-Is (shear+torsion): Diagonal stress due to torsion and shear force
Sig-Ps(Max): Maximum principal stress
Sig-Ps(Min): Minimum principal stress

4.3 Check principal stress at a construction stage

$$\sigma_{ps} \le \sigma_{ta} = k_t f_{ctm}(t) \tag{1.54}$$

4.4 Verification of principal stress at a construction stage

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Principal stress at a construction stage ...

Model View | Principal stress at a construction stage

Elem	Part	Comp./Tens.	Stage	CHK	Sig_P1 (kN/m²)	Sig_P2 (kN/m²)	Sig_P3 (kN/m²)	Sig_P4 (kN/m²)	Sig_P5 (kN/m²)	Sig_P6 (kN/m²)	Sig_P7 (kN/m²)	Sig_P8 (kN/m²)	Sig_P9 (kN/m²)	Sig_P10 (kN/m²)	Sig_MAX (kN/m²)	Sig_AP (kN/m²)
1	I[1]	Tension	CS02	NG	-22022.845	-312.0054	-739.4461	-13230.135	-4314.0122	-1095.0335	-5140.6831	-1368.9156	-4585.9622	-1188.4212	-22022.845	2114.8326
1	J[2]	Tension	CS02	OK	-985.3343	-462.7667	-873.7914	-1543.1593	622.0289	100.6738	421.2502	-50.1476	477.7564	-62.0946	-1543.1593	2114.8326
10	I[10]	Tension	CS02	OK	-35.4521	-33.7141	-19.0499	-19.5566	-682.0149	162.4717	-314.0479	260.9484	210.9158	482.1331	-682.0149	2114.8326
10	J[11]	Tension	CS01	OK	-0.0000	-0.0000	-1245.4446	-1245.4446	466.3484	466.3484	306.0943	306.0943	-227.0447	-227.0447	-1245.4446	2114.8326

[Fig. 1.41] Result table for principal stress at a construction stage

Elem: Element number.
Part: Check location (I-End, J-End) of each element.
Comp./Tens.: Compression or Tension Stress.
Stage: Construction stage.
CHK: Principal stress check for construction stages.
Sig_P1: Principal Stress at the left top of top flange.
Sig_P2: Principal Stress at the right top of top flange.
Sig_P3: Principal Stress at the right bottom of bottom flange.
Sig_P4: Principal Stress at the left bottom of bottom flange.
Sig_P5: Principal Stress at the top of left web.(at Z1 Level)
Sig_P6: Principal Stress at the top of right web.(at Z1 Level)
Sig_P7: Principal Stress at the neutral axis in left web.(at Z2 Level)
Sig_P8: Principal Stress at the neutral axis in right web.(at Z2 Level)
Sig_P9: Principal Stress at the bottom of left web.(at Z3 Level)
Sig_P10: Principal Stress at the bottom of right web.(at Z3 Level)
Sig_MAX: The maximum Principal stress among P1-P10.
Sig_AP: Allowable principal stress at neutral axis in the web.

5. Principal stress at service loads

Verify the principal tensile stress at the stress verification point 1~10 defined in Section Manager dialog box.

Maximum principal stress under the serviceability load combination ≤ Allowable stress

5.1 Allowable tensile stress

$$\sigma_{ca} = k_t f_{ctm} \tag{1.55}$$

where,

k_t : *If "User Input Data" option is checked on, the coefficient of "k_t" will be applied as the user defined value. However, if the option is checked off, "0.6" will be applied.*

f_{ctm} : *The mean compressive strength at 28 days. Refer to the clause 1.1 for the calculation of "f_{ctm}"*

5.2 Maximum principal stress

The maximum principal stress at the service state can be calculated as the following equation.

$$\sigma_{ps} = \frac{1}{2}\left[\left(\sigma_x + \sigma_z\right) \pm \sqrt{\left(\sigma_x - \sigma_z\right)^2 + 4\left(\tau_s + \tau_t + \tau_p\right)^2}\right] \tag{1.56}$$

where,

σ_x : *Sum of axial stresses in ECS x-direction*

σ_z : *Sum of axial stresses in ECS z-direction*

τ_s : *Shear stress due to shear.*

τ_t : *Shear stress due to torsion.*

τ_p : *Shear stress due to shear reinforcement.*

Beam stresses of PSC

Stress components to calculate the maximum principal stress can be checked in the table below.

Results>Result Tables>Beam>Stress(PSC)…

Model View | Result-[Beam Stress(PSC)]

Elem	Load	Stage	Step	Part	Section Position	Sig-xx(Axial) (kN/m²)	Sig-xx(Moment-y)	Sig-xx(Moment-z)	Sig-xx(Bar) (kN/m²)	Sig-xx(Summation)	Sig-zz (kN/m²)	Sig-xz (shear)	Sig-xz (torsion)	Sig-xz (bar) (kN/m²)	Sig-Is (shear) (kN/m²)	Sig-Is (shear+torsio	Sig-Ps(Max) (kN/m²)	Sig-Ps(Min) (kN/m²)
1	Summati	CS02	001(first	I[1]	Pos-1	6.8765e+000	-7.1923e+00	2.8765e+004	-9.8251e+00	2.1481e+004	0.0000e+000	0.0000e+000	3.4540e+003	0.0000e+000	2.1481e+004	2.2023e+004	2.2023e+004	-5.4172e+00
1	Summati	CS02	001(first	I[1]	Pos-2	6.8765e+000	-7.1923e+00	-3.0641e+00	-9.8251e+00	-3.7925e+00	0.0000e+000	0.0000e+000	-3.4540e+00	0.0000e+000	0.0000e+000	3.1201e+002	3.1201e+002	-3.8237e+00
1	Summati	CS02	001(first	I[1]	Pos-3	6.8765e+000	-5.0342e+00	-1.4800e+00	-9.8251e+00	-1.5394e+00	0.0000e+000	0.0000e+000	-3.4540e+00	0.0000e+000	0.0000e+000	7.3945e+002	7.3945e+002	-1.6134e+00
1	Summati	CS02	001(first	I[1]	Pos-4	6.8765e+000	-5.0342e+00	1.2923e+004	-9.8251e+00	1.2328e+004	0.0000e+000	0.0000e+000	3.4540e+003	0.0000e+000	1.2328e+004	1.3230e+004	1.3230e+004	-9.0174e+00
1	Summati	CS02	001(first	I[1]	Pos-5	6.8765e+000	-2.5078e+00	3.4182e+003	-9.8251e+00	3.0761e+003	-1.5441e+00	1.3048e+002	3.4540e+003	-8.9151e+00	3.1982e+003	4.3140e+003	4.3140e+003	-2.7821e+00
1	Summati	CS02	001(first	I[1]	Pos-6	6.8765e+000	-2.5078e+00	-5.2947e+00	-9.8251e+00	-5.6368e+00	-1.5441e+00	1.3048e+002	-3.4540e+00	-8.9151e+00	-1.4072e+00	1.0950e+003	1.0950e+003	-8.2760e+00
1	Summati	CS02	001(first	I[1]	Pos-7	6.8765e+000	7.1302e+002	3.4182e+003	-9.8251e+00	4.0399e+003	-1.5441e+00	1.5022e+002	3.4540e+003	-8.9151e+00	4.1366e+003	5.1407e+003	5.1407e+003	-2.6450e+00
1	Summati	CS02	001(first	I[1]	Pos-8	6.8765e+000	7.1302e+002	-5.2947e+00	-9.8251e+00	-4.6730e+00	-1.5441e+00	1.5022e+002	-3.4540e+00	-8.9151e+00	-1.3774e+00	1.3689e+003	1.3689e+003	-7.5861e+00
1	Summati	CS02	001(first	I[1]	Pos-9	6.8765e+000	7.5333e+001	3.4182e+003	-9.8251e+00	3.4022e+003	-1.5441e+00	1.3134e+002	3.4540e+003	-8.9151e+00	3.5164e+003	4.5860e+003	4.5860e+003	-2.7279e+00
1	Summati	CS02	001(first	I[1]	Pos-10	6.8765e+000	7.5333e+001	-5.2947e+00	-9.8251e+00	-5.3107e+00	-1.5441e+00	1.3134e+002	-3.4540e+00	-8.9151e+00	-1.3965e+00	1.1884e+003	1.1884e+003	-8.0433e+00
1	Summati	CS02	001(first	J[2]	Pos-1	-8.7906e+00	-7.7820e+00	6.4075e+003	-9.5788e+00	-1.0261e+00	0.0000e+000	0.0000e+000	3.3289e+003	0.0000e+000	0.0000e+000	9.8533e+002	9.8533e+002	-1.1246e+00
1	Summati	CS02	001(first	J[2]	Pos-2	-8.7906e+00	-7.7820e+00	-6.8146e+00	-9.5788e+00	-2.3483e+00	0.0000e+000	0.0000e+000	-3.3289e+00	0.0000e+000	0.0000e+000	4.6277e+002	4.6277e+002	-2.3946e+00
1	Summati	CS02	001(first	J[2]	Pos-3	-8.7906e+00	3.6708e+002	-3.2887e+00	-9.5788e+00	-1.1808e+00	0.0000e+000	0.0000e+000	-3.3289e+00	0.0000e+000	0.0000e+000	8.7379e+002	8.7379e+002	-1.2682e+00
1	Summati	CS02	001(first	J[2]	Pos-4	-8.7906e+00	3.6708e+002	2.8816e+003	-9.5788e+00	-5.6378e+00	0.0000e+000	0.0000e+000	3.3289e+003	0.0000e+000	0.0000e+000	1.5432e+003	1.5432e+003	-7.1809e+00
1	Summati	CS02	001(first	J[2]	Pos-5	-8.7906e+00	-6.5942e+00	7.6604e+002	-9.5788e+00	-8.7798e+00	-1.5441e+00	3.0536e+002	3.3289e+003	-8.9151e+00	-1.4970e+00	-6.2203e+00	-6.2203e+00	-9.7019e+00
1	Summati	CS02	001(first	J[2]	Pos-6	-8.7906e+00	-6.5942e+00	-1.1732e+00	-9.5788e+00	-1.0719e+00	-1.5441e+00	3.0536e+002	-3.3289e+00	-8.9151e+00	-1.5069e+00	-1.0067e+00	-1.0067e+00	-1.2163e+00
1	Summati	CS02	001(first	J[2]	Pos-7	-8.7906e+00	7.4662e+002	7.6604e+002	-9.5788e+00	-7.3738e+00	-1.5441e+00	3.5675e+002	3.3289e+003	-8.9151e+00	-1.4955e+00	-4.2125e+00	-4.2125e+00	-8.4967e+00
1	Summati	CS02	001(first	J[2]	Pos-8	-8.7906e+00	7.4662e+002	-1.1732e+00	-9.5788e+00	-9.3130e+00	-1.5441e+00	3.5675e+002	-3.3289e+00	-8.9151e+00	-1.5075e+00	5.0148e+001	5.0148e+001	-1.0907e+00
1	Summati	CS02	001(first	J[2]	Pos-9	-8.7906e+00	5.8381e+002	7.6604e+002	-9.5788e+00	-7.5366e+00	-1.5441e+00	3.0627e+002	3.3289e+003	-8.9151e+00	-1.4875e+00	-4.7776e+00	-4.7776e+00	-8.6030e+00
1	Summati	CS02	001(first	J[2]	Pos-10	-8.7906e+00	5.8381e+002	-1.1732e+00	-9.5788e+00	-9.4758e+00	-1.5441e+00	3.0627e+002	-3.3289e+00	-8.9151e+00	-1.5012e+00	6.2095e+001	6.2095e+001	-1.1082e+00

[Fig. 1.42] Result table for beam stress

Sig-xx (Axial): Axial stress due to the axial force (Fx) in the ECS x-direction

Sig-xx (Moment-y): Stress due to My (moment about the ECS y-axis) in ECS x-direction

Sig-xx (Moment-z): Stress due to Mz (moment about the ECS z-axis) in ECS x-direction

Sig-xx (Bar): Axial stress due to shear steel bars in the ECS x-direction

Sig-xx (Summation): Sum of the axial stress in the ECS x-direction and the axial stress due to shear steel bars in the ECS x-direction

Sig-zz: Stress in the ECS z-direction

Sig-xz (shear): Sum of shear stresses due to shear force and shear steel bars

Sig-xz (torsion): Shear stress due to torsion

Sig-xz (bar): Shear stress due to shear steel bars

Sig-Is (shear): Diagonal stress due to shear force

Sig-Is (shear+torsion): Diagonal stress due to torsion and shear force

Sig-Ps(Max): Maximum principal stress

Sig-Ps(Min): Minimum principal stress

5.3 Check principal stress at a service loads

$$\sigma_{ps} \le \sigma_{ta} = k_t f_{ctm} \tag{1.57}$$

5.4 Verification of principal stress at a service loads

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Principal stress at a service loads …

Model View | Principal stress at service loads

Elem	Part	Comp./Tens.	LCom Name	Type	CHK	Sig_P1 (kN/m²)	Sig_P2 (kN/m²)	Sig_P3 (kN/m²)	Sig_P4 (kN/m²)	Sig_P5 (kN/m²)	Sig_P6 (kN/m²)	Sig_P7 (kN/m²)	Sig_P8 (kN/m²)	Sig_P9 (kN/m²)	Sig_P10 (kN/m²)	Sig_MAX (kN/m²)	Sig_AP (kN/m²)
1	I[1]	Tension	cLCB13	FX-MAX	OK	-37817.3784	-193.1592	-393.9074	-17350.2088	-5362.0687	-1389.5139	-5298.5503	-1454.5032	-5359.3061	-1392.3385	-37817.3784	2105.2928
1	J[2]	Tension	cLCB13	FX-MIN	OK	-28642.3678	-232.5405	-485.7094	-13774.4309	-4654.5931	-1191.9129	-4783.1795	-1236.7099	-4959.2239	-1254.5937	-28642.3678	2105.2928
10	I[10]	Tension	cLCB13	FX-MIN	OK	-12244.4707	-0.0209	-0.0010	-2028.5512	-9696.3266	-67.7835	-6943.5875	-58.0471	-3176.9489	-18.6744	-12244.4707	2105.2928
10	J[11]	Tension	cLCB13	FX-MIN	OK	-15699.0620	-0.0889	-0.0201	-2790.7183	-12573.6215	-67.3872	-9041.7919	-55.6472	-4126.7435	-19.0837	-15699.0620	2105.2928

[Fig. 1.43] Result table for principal stress at a service loads

Elem: Element number.

Part: Check location (I-End, J-End) of each element.

Comp./Tens.: Compression or Tension Stress.

LCom. Name: Load combination name.

Type : Displays the set of member forces corresponding to moving load case or settlement load case for which the maximum stresses are produced

CHK: Principal stress check for service loads at maximum shear force.

Sig_P1: Principal Stress at the left top of top flange.

Sig_P2: Principal Stress at the right top of top flange.

Sig_P3: Principal Stress at the right bottom of bottom flange.

Sig_P4: Principal Stress at the left bottom of bottom flange.

Sig_P5: Principal Stress at the top of left web.(at Z1 Level)

Sig_P6: Principal Stress at the top of right web.(at Z1 Level)

Sig_P7: Principal Stress at the neutral axis in left web.(at Z2 Level)

Sig_P8: Principal Stress at the neutral axis in right web.(at Z2 Level)

Sig_P9: Principal Stress at the bottom of left web.(at Z3 Level)

Sig_P10: Principal Stress at the bottom of right web.(at Z3 Level)

Sig_MAX: The maximum Principal stress among P1-P10.

Sig_AP: Allowable principal stress at neutral axis in the web.

6. Check crack width

Cracking shall be limited to satisfy the following condition.

Crack width, $w_k \leq$ Crack width limit, w_{max}

6.1 Calculate crack widths

(1) Determine ε_{sm}-ε_{cm}

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1+\alpha_e \rho_{p,eff}\right)}{E_s} \geq 0.6\frac{\sigma_s}{E_s} \quad (1.58)$$

EN1992-1-1:2004 (7.9)

where,

ε_{sm} : *The mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tensile stiffening.*

ε_{cm} : *The mean strain in the concrete between cracks.*

σ_s : *The stress in the tension reinforcement assuming a cracked section.*

α_e : *The ratio of* E_s/E_{cm}.

E_s : *The design value of modulus of elasticity of reinforcing steel.*

E_{cm} : *The secant modulus of elasticity of concrete.*

$$f_{ct,eff} = f_{ctm} \quad (1.59)$$

EN1992-1-1:2004 (7.10)

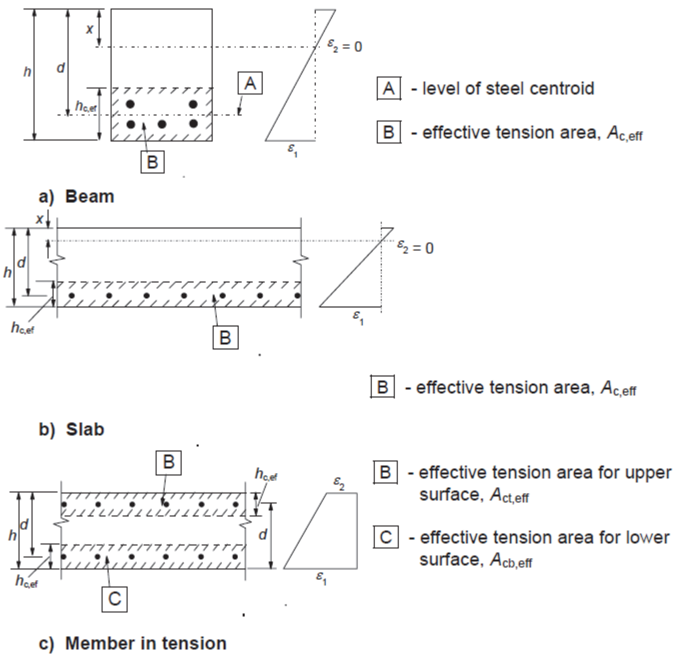
$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A_p'}{A_{c,eff}} \quad (1.60)$$

A_p' : *The area of pre or post-tensioned within* $A_{c,eff}$.

$A_{c,eff}$: *The effective area of concrete in tension surrounding the reinforcement of prestressing tendons of depth,* $h_{c,ef}$.

$$h_{c,ef} = \min\left[2.5\left(h-d\right), \quad \frac{h-x}{3}, \quad \frac{h}{2}\right] \quad (1.61)$$

EN1992-1-1:2004 7.3.2(3)



[Fig. 1.44] Effective tension area (typical cases)

EN1992-1-1:2004 Figure 7.1

$$\xi_1 = \sqrt{\xi \frac{\phi_s}{\phi_p}} \quad (1.62)$$

If only prestressing steel is used to control cracking, $\xi_1=\sqrt{\xi}$

ξ : the ratio of bond strength of prestressing and reinforcing steel.

[Table 1.26] Ratio of bond strength, ξ

EN1992-1-1:2004 Table 6.2

Prestressing Steel	ξ		
	Pre-tensioned	Bonded, post-tensioned	
		≤ C50/60	≥ C70/80
Smooth bars and wires	Not applicable	0.3	0.15
Strands	0.6	0.5	0.25
Indented wires	0.7	0.6	0.3
Ribbed bars		0.7	0.35

φ_s : The largest bar diameter of reinforcing steel.
φ_p : The equivalent diameter of tendon.
k_t : A factor dependent on duration of the load.

[Table 1.27] Factor k

EN1992-1-1:2004 7.3.4(2)

Condition	k_t
Short term loading	0.6
Long term loading	0.4

- Definition of Short and Long term loads

[Table 1.28] Definition of duration of the load

Condition	Description
Long term loading	Load combination composed of long-term load cases only
Short term loading	Load combinations other than long-term load combination

When the user does not define the long-term or short-term load case, it will be classified as shown in the following table.

[Table 1.29] Classification for duration of the load

Duration of the load	Description
Long term load case	Following static load case D : Dead Load DC : Dead Load of Component and Attachments. DW : Dead Load of Wearing Surfaces and Utilities. L : Live Load. LR : Roof Live Load.
Short term load case	Load cases other than long-term load cases

(2) Determine $s_{r,max}$

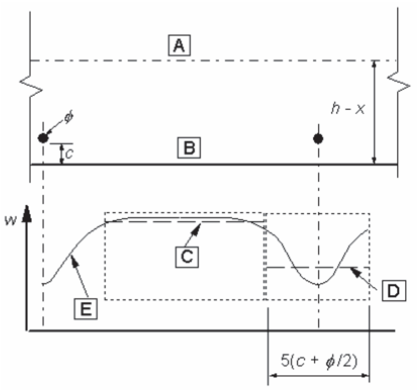
The maximum crack spacing, $s_{r,max}$ is calculated as shown in the table below.

[Table 1.30] Maximum crack spacing, $s_{r,max}$

Condition	$s_{r,max}$
Spacing ≤ 5(c+ϕ/2)	$k_3 c + \frac{k_1 k_2 k_4 \phi}{\rho_{p,eff}}$
Spacing > 5(c+ϕ/2) or No bonded reinforcement	$1.3(h-x)$

EN1992-1-1:2004 (7.12)

EN1992-1-1:2004 (7.14)



[Fig. 1.45] Crack width, w, at concrete surface relative to distance from bar

EN1992-1-1:2004 Figure 7.2

where,

φ : The bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, φ_{eq}, *should be used.*

For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2.

$$\phi_{eq} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2} \quad (1.63)$$

c : The cover to the longitudinal reinforcement.

k_1 *: A coefficient which takes account of the bond properties of the bonded reinforcement.*

[Table 1.31] Coefficient k_1

Condition	k_1
High bond bars	0.8
Bars with an effectively plan surface	1.0

EN1992-1-1:2004 7.3.4(2)

k_2 *: A coefficient which takes account of the distribution of strain.*

[Table 1.32] Coefficient k_2

Condition	k_2
Bending	0.5
Pure Tension	1.0

k_3, k_4 *: If "User Input Data" option is checked on, the coefficient of "*k_3 *and* k_4*" will be applied as the user defined value. However, if the option is checked off, it will be applied as the following value.*

$k_3 = 3.4$

$k_4 = 0.425$

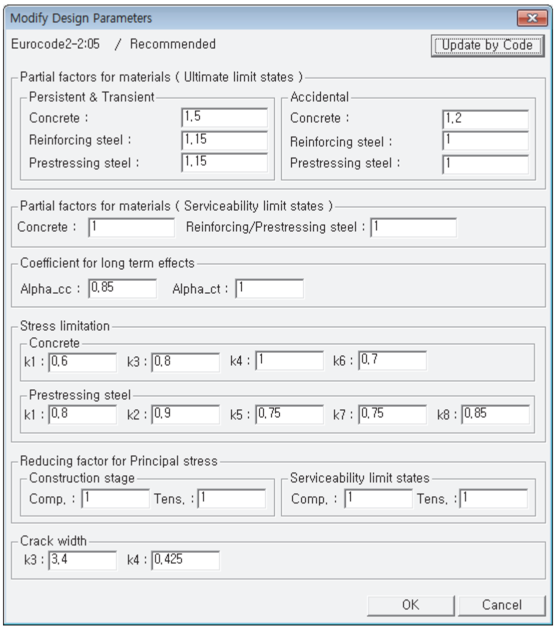
(3) Calculate the design crack width, w_k

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{1.64}$$

EN1992-1-1:2004 (7.8)

Coefficient k_3, k_4 for crack

Design>PSC Design>PSC Design Parameters...



[Fig. 1.46] Input coefficient k3, k4 for crack

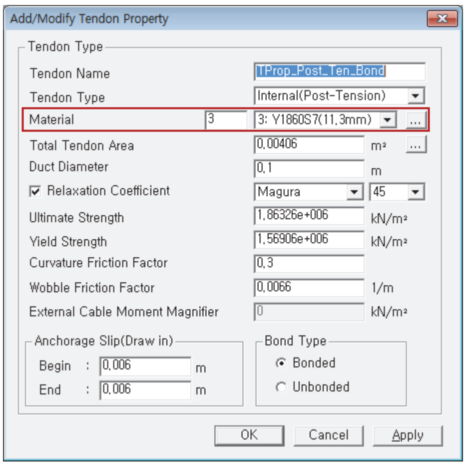
Prestressing steel type for ξ

In midas Civil, the following prestressing steel types are available.

[Table 1.33] Prestressing steel type supported in program

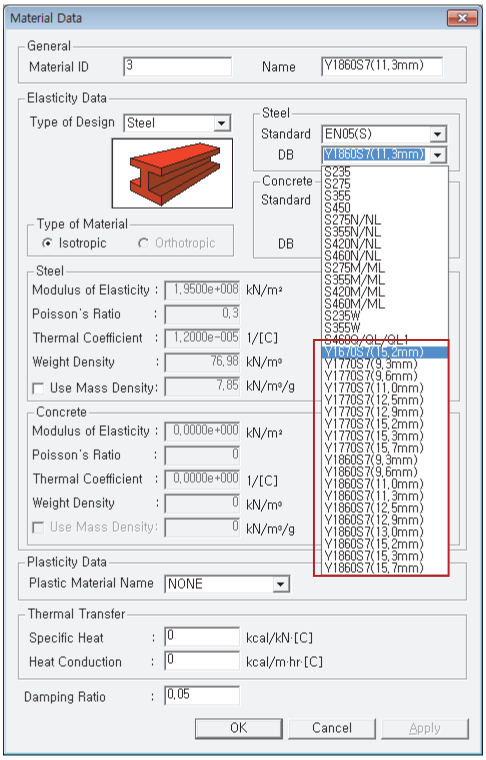
Prestressing Steel	Description
Smooth bars and wires	Other than Strands
Strands	When the material properties of tendon is specified as follows: Standard = EN05(S) DB = Y1670 Series, Y1770 Series, Y1860 Series

Load>Prestress Loads>Tendon Property...



[Fig. 1.47] Define material of tendon

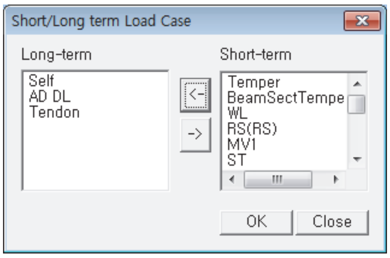
- *Model>Properties>Material...*



[Fig. 1.48] Steel material list of “EN05(S)” standard

Duration of load (Short/Long term)

- *Design>Common Parameter>Short/Long term Load Case*



[Fig. 1.49] Define short/long term load case

6.2 Get a limiting calculated crack width, w_{max}

(1) Recommended values of w_{max} (mm)

EN1992-1-1:2004
Table 7.1N

[Table 1.34] Limiting crack width, w_{max}

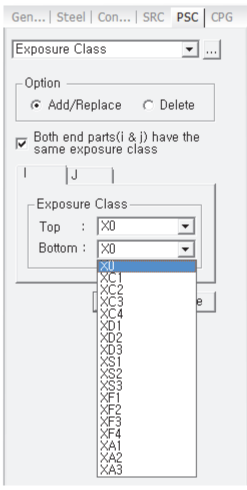
Exposure Class	Unbonded		Bonded		Others
	Quasi	Frequent	Quasi	Frequent	
X0	0.3	Not Checked	Not Check	0.2	Not Check
XC1					
XC2	0.3	Not Checked	0.0 (Decom-Pression)	0.2	Not Checked
XC3					
XC4					
XD1			Not Check	Decom-pression	Not Checked
XD2					
XD3					
XS1	0.3	Not Checked	Not Checked	0.0 (Decom-pression)	Not Checked
XS2					
XS3					
XF1*	0.3	Not Checked	Not Checked	0.3	Not Checked
XF2*					
XF3*					
XF4*					
XA1*					
XA2*					
XA3*					

(*) In midas Civil, the limit value of "Freeze/Thaw attack class(XF1~XF4) and Chemical attack class(XA1~XA3)" is applied as "0.3mm".

Exposure Class

Exposure class can be defined by members in the following dialog box.

Design>PSC Design>Exposure Class...



[Fig. 1.50] Define exposure class for crack

6.3 Check crack width at service loads

$$w_k \le w_{\max} \tag{1.65}$$

6.4 Verification of crack width at service loads

By Result Tables

The design results can be checked as shown in the table below.

Design>PSC Design>PSC Design Result Tables>Check crack width at service loads…

Model View | Check crack width at service loads

Elem	Part	Top/Bottom	LCom Name	Serviceability Load Type	Type	CHK	M_Ed (kN·m)	Sig_T (kN/m²)	Sig_B (kN/m²)	Wk (m)	Wmax (m)
1	I[1]	Bottom	cLCB17	Frequent	FX-MIN	OK	33.2276	0.3945	-0.9054	0.0000	0.0002
1	I[1]	Top	cLCB13	Frequent	FX-MAX	OK	0.0371	0.0000	-0.0000	0.0000	0.0002
1	J[2]	Bottom	cLCB17	Frequent	FX-MAX	OK	1128.6860	12.0763	-27.9111	0.0000	0.0002
1	J[2]	Top	cLCB13	Frequent	FX-MAX	OK	880.0597	0.0000	-0.0000	0.0000	0.0002
10	I[10]	Bottom	cLCB17	Frequent	FX-MAX	NG	3441.2994	13.1802	-28.7671	0.0000	0.0000
10	I[10]	Top	cLCB13	Frequent	FX-MIN	OK	-3752.3285	-38.5008	14.2238	0.0000	0.0002
10	J[11]	Bottom	cLCB17	Frequent	FX-MAX	NG	3072.9755	12.1664	-29.0184	0.0000	0.0000
10	J[11]	Top	cLCB13	Frequent	FX-MIN	OK	-4919.9422	-46.1173	16.2866	0.0000	0.0002

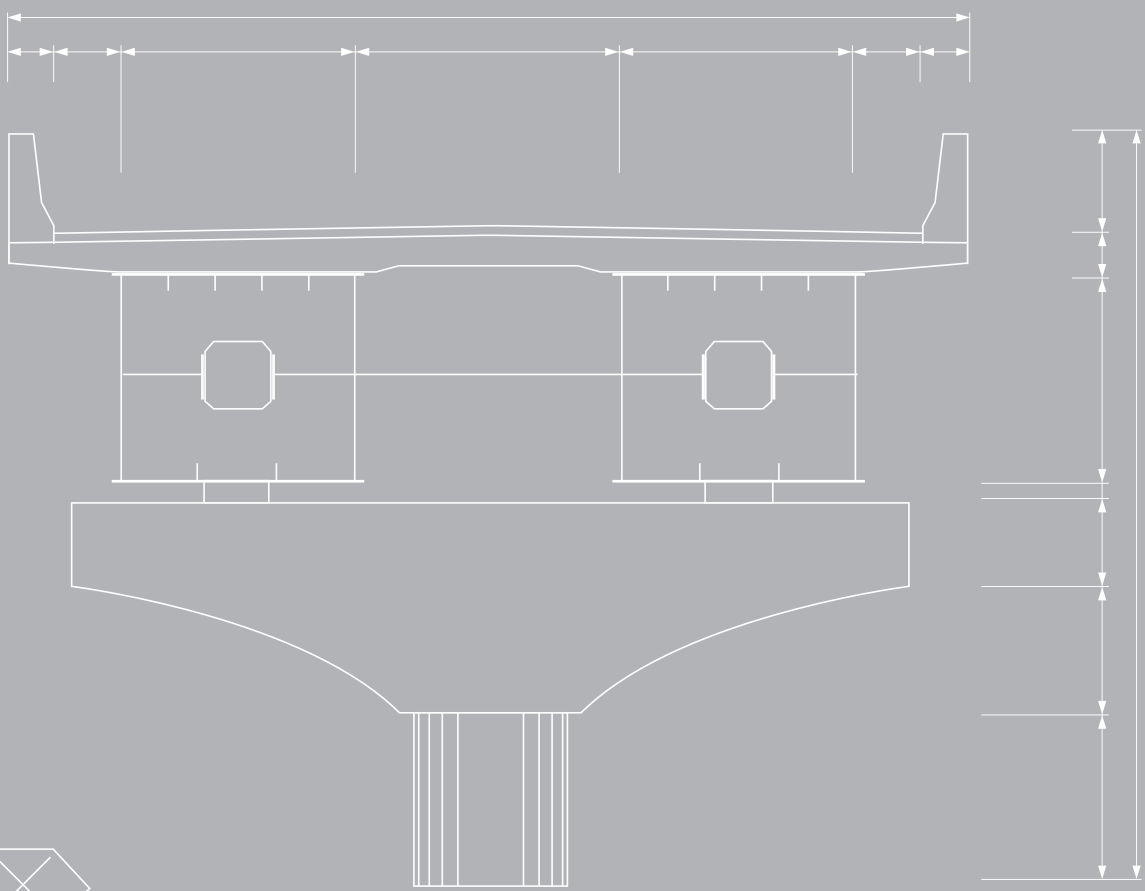
[Fig. 1.51] Result table for crack width at service loads

Elem: Element number
Part: Check location (I-End, J-End) of each element
Top/Bottom: At top of element, at bottom of element
LCom. Name: Load combination name.
Serviceability Load Type : Frequent/ Quasi-Static
Type: produce maximum and minimum member force components for the load combinations including moving load cases or settlement load cases.
Check:OK/NG
M_Ed : Maximum Moment in the Section.
Sig_T : Stress at the top.
Sig_B : Stress at the bottom.
wk : Crack width
wmax : Allowable crack limit

Chapter 2.

Composite Steel Box Girder Design

EN 1994-2



Chapter 2.

Composite Steel Box Girder Design (EN 1994-2)

Composite steel box girder needs to be designed to satisfy the following limit states.

Ultimate Limit States

- Bending Resistance
- Resistance to Vertical Shear
- Resistance to Longitudanal Shear
- Resistance to Fatigue

Serviceability Limit States

Stress limitation for concrete	For Quasi-permanent, Characteristic Load Combination
Stress limitation for reinforcing steel	For Quasi-permanent, Characteristic, Frequent Load Combination
Stress limitation for structral steel	For Characteristic Load Combination
Resistance to Longitudinal Shear	For Quasi-permanent, Characteristic, Frequent Load Combination

Ultimate Limit States

1. Bending resistance

Limit state of Bending Resistance will satisfy the condition, $M_{Ed} \le M_{Rd}$.
Moment resistance, M_{Rd}, shall be calculated as follows:

1.1 Design values of material

(1) Partial factors for materials

Partial factor for materials considered in ultimate limit states are shown in the table below.

In midas Civil, partial factor for materials can be specified by the user in "Design Parameter" dialog box. The default values are determined as below as per Eurocode 4.

EN1994-2:2005 2.4.1.2

[Table 2.1] Partial factor for materials

Materials	Condition	Partial Factor
Concrete	Persistent & Transient	γ_c = 1.5
	Accidental	γ_c = 1.2
Reinforcing steel	Persistent & Transient	γ_s = 1.15
	Accidental	γ_s = 1.0
Structural steel	Cross-sections	γ_{M0} = 1.0
	Members to instability assessed	γ_{M1} = 1.0
Shear connection	members to instability	γ_V = 1.25
Fatigue verification of headed studs	Strength	γ_{Mf} = 1.0
	Strength of studs in shear	$\gamma_{Mf,s}$ = 1.0

(2) Design compressive strength of concrete.

$$f_{cd} = f_{ck} / \gamma_c \qquad (2.1)$$

EN1994-2:2005 (2.1)

where,
f_{ck} : The characteristic compressive cylinder strength of concrete at 28 days.
γ_c : The partial safety factor for concrete.

(3) Design yield strength of steel reinforcement.

$$f_{sd} = f_{sk} / \gamma_s \qquad (2.2)$$

where,
f_{sk} : The characteristic value of the yield strength of reinforcing steel.
γ_s : The partial factor for reinforcing steel.

(4) Design yield strength of structural steel.

$$f_{yd} = f_y / \gamma_{M0} \tag{2.3}$$

where,

f_y : *The nominal value of the yield strength of structural steel.*

γ_{M0} : *The partial factor for structural steel applied to resistance of cross-sections.*

The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel shall be obtained by using the simplification given in Fig. 2.1.

EN1993-1-1:2005
Table 3.1

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \le 40$ mm		40 mm $< t \le 80$ mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

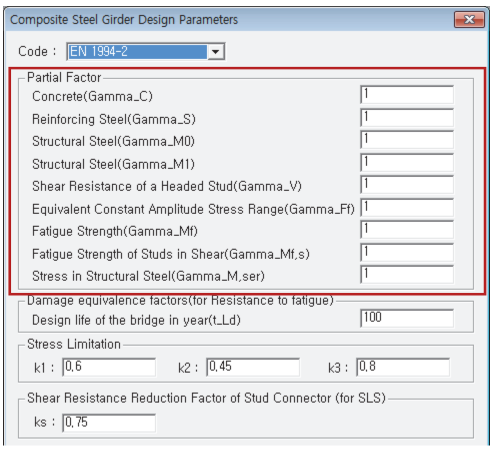
[Fig. 2.1] Nominal values of yield strength f_y and ultimate tensile strength f_u

Partial safety factor

Parameters related to the material such as partial factors, damage equivalence factors, and shear resistance reduction factor can be defined in "Composite Steel Girder Design Parameters" dialog box.

The default values of partial factors are defined as "1.0".

Design > Composite Steel Girder Design>Design Parameters…



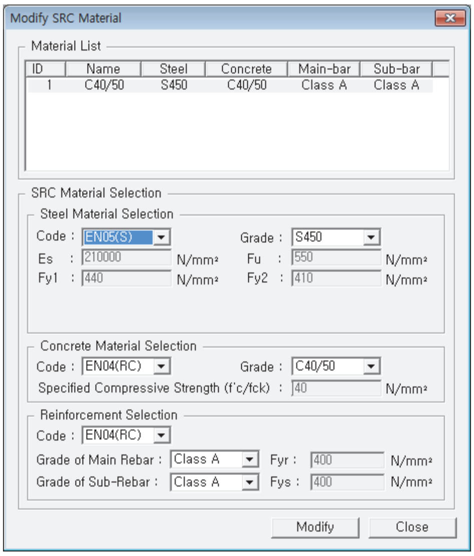
[Fig. 2.2] Design Parameters Dialog

Design strength of materials

Design strength of concrete, reinforcement, and steel can be defined in “Modify SRC Material” dialog box.

In Steel Design Selection field, when Code is entered as “EN05”, F_{y1} is tensile strength of the steel for which the thickness is less or equal to 40mm and F_{y2} is tensile strength of the steel for which the thickness is larger than 40mm.

Design > Composite Steel Girder Design>Design Material…



[Fig. 2.3] Composite steel girder design material

1.2 Classification of cross-section

The classification system defined in EN1993-1-1:2005, 5.5.2 applies to cross-sections of composite beams.

EN1993-1-1:2005
5.5.2

[Table 2.2] Classes of cross-sections

Class	Defined as
1	which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance
2	which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling
3	in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance
4	in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section

(1) The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression.

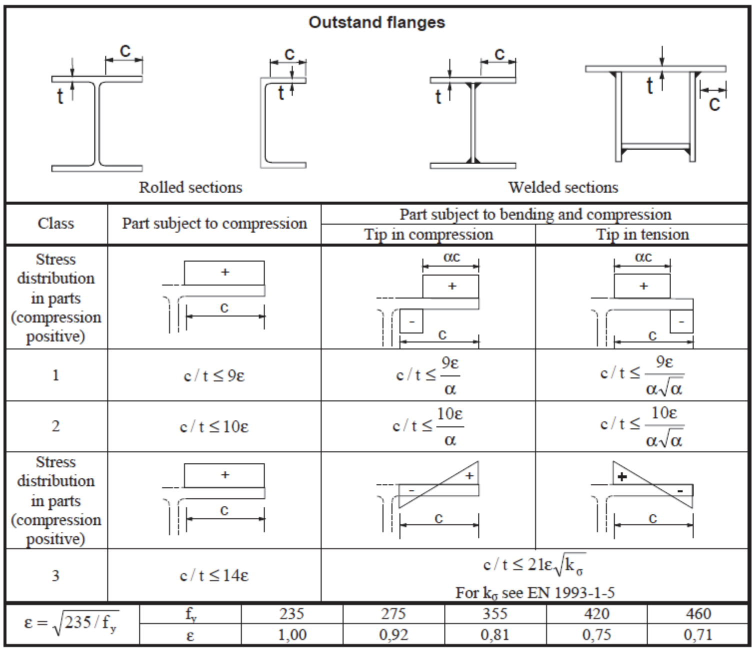
• Classification of Class in flange

Class of flange can be classified depending on the positive and negative moment.

[Table 2.3] Class of compression flange

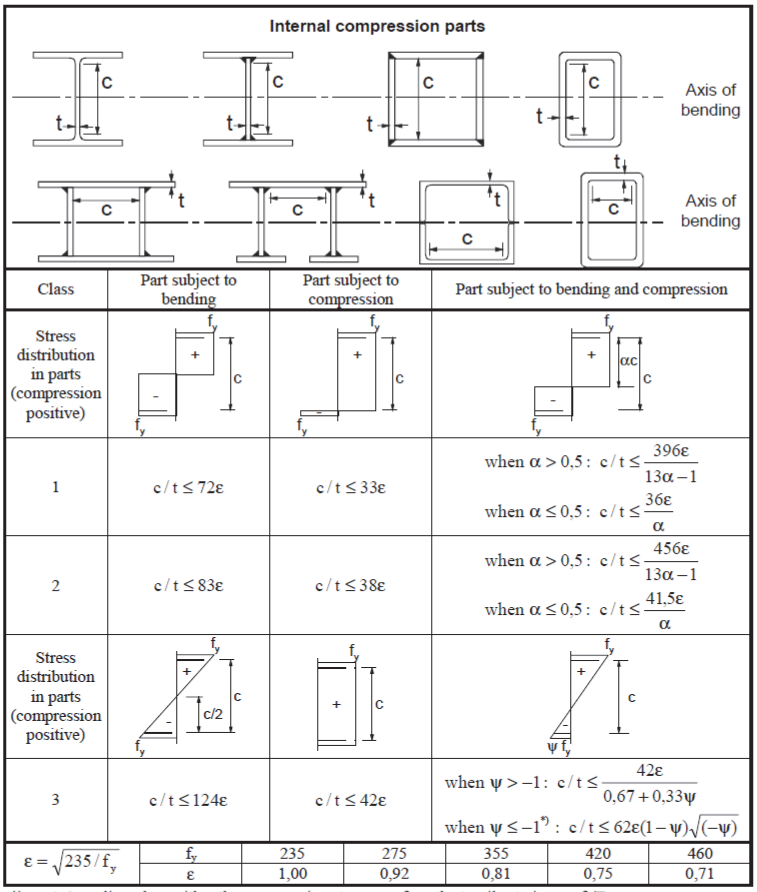
Moment	Position	Class of compression flange
Positive	Top Flange	A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class1.
Negative	Bottom Flange	Composite-I : Check for outstand flanges in Fig.2.4. Composite-Box : Check for outstand flanges and internal compression part in Fig.2.5.

EN1993-1-1:2005
Table 5.2



[Fig. 2.4] Maximum width-to-thickness ratios for compression parts - Outstand

• Classification of Class in web: Check for internal compression part in Fig. 2.5.



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0,5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0,5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5$: $c/t \le \frac{41,5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0,67 + 0,33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y/E$

[Fig. 2.5] Maximum width-to-thickness ratios for compression parts - Internal

EN1993-1-1:2005 Table 5.2

(2) Classification of a cross-section : A cross-section is classified according to the highest (least favorable) class of its compression parts as follows.

EN1993-1-1:2005 5.5.2(6)

[Table 2.4] Class of section according to class of compression parts

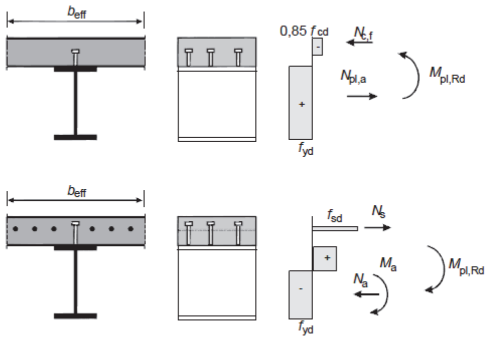
Class of Section		Class of Flange			
		1	2	3	4
Class of Web	1	1	2	3	4
	2	1	2	3	4
	3	3	3	3	4
	4	4	4	4	4

* : Cross-sections with webs in Class3 and flanges in Class1 or 2 may be treated as an effective cross-sections in Class2 with an effective web in accordance with EN1993-1-1:2005, 6.2.2.4. (This clause is applied to box girder.)

EN1993-1-1:2005 5.5.2(11)

1.3 Calculate plastic bending resistance, $M_{pl,Rd}$.

- For positive moment: Compressive rebar in the deck will be ignored.
- For negative moment: Concrete area of deck will be neglected and only the tensile rebar in the deck will be considered.

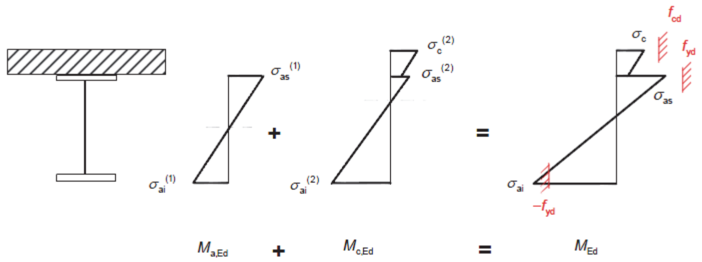


[Fig. 2.6] Plastic stress distributions for a composite beam

EN1993-1-1:2005 Figure 6.2

1.4 Calculate elastic bending resistance, $M_{el,Rd}$

$$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \tag{2.4}$$



[Fig. 2.7] Calculation of $M_{el,Rd}$

where,

$M_{a,Ed}$: *The design bending moment applied to structural steel section before composite behavior. Bending moment obtained during the construction stage analysis is used in midas Civil.*

$M_{c,Ed}$: *The part of design bending moment acting on the composite section. Bending moment obtained from the final construction stage is used in midas Civil.*

k : *The lowest factor such that a stress limit in EN1994-2:2005, 6.2.1.5(2) is reached. In midas Civil, the value of "k" is calculated as below.*

[Table 2.5] Calculation of k

Type	For Positive Moment	For Negative Moment
Steel Girder	$k_a = \dfrac{f_{yd} - M_{a,Ed}(z_a / I_{y,a})}{M_{c,Ed}(z_c / I_{y,c})}$	$k_a = \dfrac{f_{yd} - M_{a,Ed}(z_a / I_{y,a})}{M_{c,Ed}(z_c / I_{y,c})}$
Slab	$k_c = \dfrac{f_{cd}}{M_{c,Ed}(z_{c,slab} / I_{y,c,slab})}$	-
Reinforcement	-	$k_s = \dfrac{f_{sd}}{M_{c,Ed}(z_{c,bar} / I_{y,c,bar})}$
k	min[k_a, k_c]	min[k_a, k_s]

1.5 Calculate effective cross-section for Class 4 section

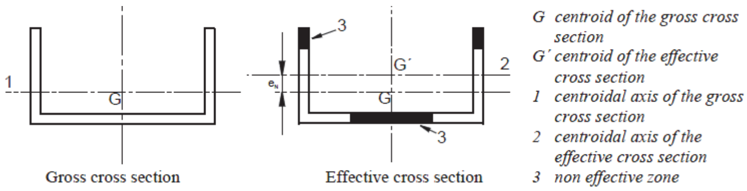
(1) Calculate effective cross-section

For cross-sections in Class4, the effective structural steel section should be determined in accordance with EN1993-1-5, 4.3.

In midas Civil, the effect of share lag is not considered in the calculation of effective area. Only the plate buckling effect is considered.

• The effective area A_{eff} should be determined assuming that the cross section is subject only to stresses due to uniform axial compression.

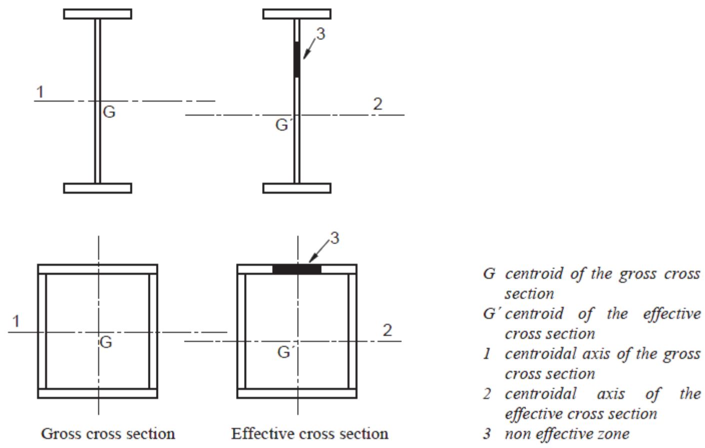
EN1993-1-5:2006 Figure 4.1



[Fig. 2.8] Class 4 cross-sections - axial force

• The effective section modulus W_{eff} should be determined assuming that the cross section is subject only to bending stresses.

EN1993-1-5:2006 Figure 4.2



[Fig. 2.9] Class 4 cross-sections - bending moment

The calculation of effective area depending on the longitudinal stiffener will be explained in the clause 1.6 and 1.7 in this manual.

(2) Consideration of additional moment due to the eccentricity of gravity center between the gross area and the effective area

In case of the section with Class 4 classification under the compressive force, the additional moment due to the different gravity center between gross area and effective area is taken into account in the design moment.

EN1993-1-1:2005 6.2.2.5(4)

$$\Delta M_{Ed} = N_{Ed} e_N = N_{Ed}\left(C_{z,c} - C_{z,c,eff}\right) \tag{2.5}$$

where,

e_N : *Eccentricity between the gross area and effective area*

$C_{z,c}$: *Gravity center of the gross area*

$C_{z,c,eff}$: *Gravity center of the effective area*

1.6 Plate elements without longitudinal stiffeners

The effective areas of flat compression elements should be obtained using Table 2.7 for internal elements and Table 2.8 for outstand elements. The effective area of the compression zone of plate should be obtained from :

$$A_{c,eff} = \rho A_c \qquad (2.6)$$

EN1993-1-5:2006 4.4

where,

$A_{c,eff}$ *: The effective cross sectional area.*

A_c *: The gross cross sectional area.*

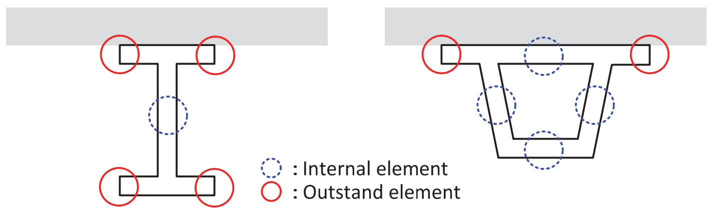
ρ *: The reduction factor for plate buckling.*

(1) Effective width b_{eff}

Refer to the following table and figure to see the definition of internal element and outstand element in midas Civil.

[Table 2. 6] Definition of internal and outstand element

Type	Shape	Defined as
Internal element	I	Web
	Box	Web / Flanges between web
Outstand element	I	Flange
	Box	Outstand flange which is the outside of webs

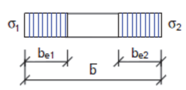
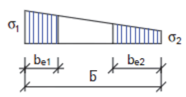
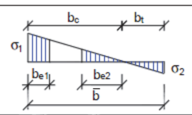


[Fig. 2.10] Internal and outstand element

- For internal compression elements

EN1993-1-5:2006 Table 4.1

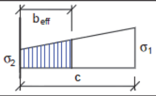
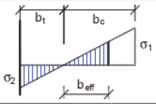
[Table 2.7] Internal compression elements

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\psi = 1$: $b_{eff} = \rho \bar{b}$ $b_{e1} = 0{,}5\, b_{eff}$ $\quad b_{e2} = 0{,}5\, b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\, b_c = \rho\, \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\, b_{eff}$ $\quad b_{e2} = 0{,}6\, b_{eff}$

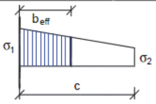
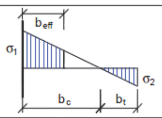
$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\,(1-\psi)^2$

• For outstand compression elements

[Table 2. 8] Outstand compression elements

Stress distribution (compression positive)	Effective^{p} width b_{eff}
	$1 > \psi \geq 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_c = \rho c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$

	$1 > \psi \geq 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_c = \rho c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578 / (\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,1\psi^2$	23,8

(2) Reduction factor ρ

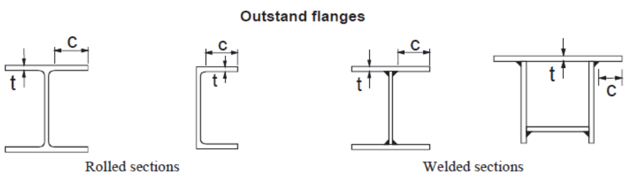
[Table 2.9] Calculation of reduction factor ρ

Type	Condition	ρ
Internal element	$\overline{\lambda}_p \leq 0.673$	1.0
	$\overline{\lambda}_p > 0.673$ where, $(3+\psi) \geq 0$	$\dfrac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^{\,2}} \leq 1.0$
Outstand element	$\overline{\lambda}_p \leq 0.748$	1.0
	$\overline{\lambda}_p > 0.748$	$\dfrac{\overline{\lambda}_p - 0.188}{\overline{\lambda}_p^{\,2}} \leq 1.0$

where,

$$\overline{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\overline{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} \tag{2.7}$$

$\overline{b}$: The appropriate width to be taken as follow.
b_s : For webs
b : For internal flange elements.
c : For outstand flanges.



[Fig. 2.11] Dimension of outstand flanges

Ψ : The stress ratio.
k_σ : The buckling factor corresponding to the stress ratio ψ and boundary conditions.
t : The thickness.
σ_{cr} : The elastic critical plate buckling stress.

$$\varepsilon = \sqrt{\frac{235}{f_y\left[N/mm^2\right]}} \tag{2.8}$$

EN1993-1-5:2006 Table 4.2

EN1993-1-5:2006 4.4(2)

EN1993-1-1:2005 Table 5.2

1.7 Stiffened plate elements with longitudinal stiffeners

The effective section area of each subpanel should be determined by a reduction factor in accordance with 1.6 to account for local buckling. The stiffened plate with effective section area for the stiffeners should be checked for global plate buckling and a reduction factor ρ should be determined for overall plate buckling.

EN1993-1-5:2006 4.5

The effective area of the compression zone of the stiffened plate should be taken as :

$$A_{c,eff} = \rho_c A_{c,eff,loc} + \sum b_{edge,eff} t \quad (2.9)$$

EN1993-1-5:2006 (4.5), (4.6)

$$A_{c,eff,loc} = A_{sl,eff} + \sum_c \rho_{loc} b_{c,loc} t \quad (2.10)$$

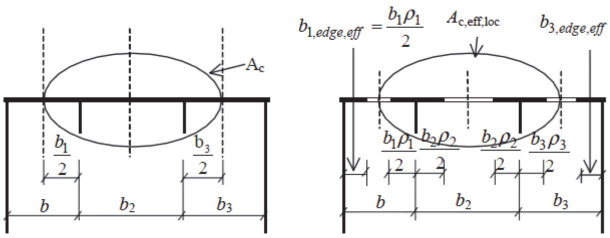
where,

$A_{c,eff,loc}$ *: The effective section areas of all the stiffeners and subpanels that are fully or partially in the compression zone except the effective parts supported by an adjacent plate element with the width* $b_{edge,eff}$.

$\sum_c$ *applies to the part of effective section of all longitudinal stiffeners with gross area* A_{sl} *located in the compression zone.*

$b_{c,loc}$ *: The width of the compressed part of each subpanel.*

ρ_{loc} *: The reduction factor for each subpanel.*



EN1993-1-5:2006 Figure 4.4

[Fig. 2.12] Stiffened plate under uniform compression

(1) Effective width and reduction factor for individual subpanels between stiffeners.
Calculate the effective width of subpanels between stiffeners as per the clause 1.6.

The value of $\bar{b}$ is taken as the smaller value between the follows:

-Clear spacing between flange and stiffener

-Clear spacing between stiffeners

(2) Elastic critical plate buckling stress $\sigma_{cr,p}$ for stiffened web.

• with single stiffener in the compression zone

$\sigma_{cr,p}$ can be calculated as follows ignoring stiffeners in the tension zone :

$$\sigma_{cr,p} = \sigma_{cr,sl} \quad (2.11)$$

EN1993-1-5:2006 A.2.2(1)

[Table 2.10] Calculation of $\sigma_{cr,sl}$

Condition	$\sigma_{cr,sl}$
$a \geq a_c$	$\frac{1.05E}{A_{sl,1}}\sqrt{\frac{I_{sl,1}t^3 b}{b_1 b_2}}$
$a < a_c$	$\frac{\pi^2 E I_{sl,1}}{A_{sl,1}a^2} + \frac{Et^3ba^2}{4\pi^2(1-\nu^2)A_{sl,1}b_1^2 b_2^2}$

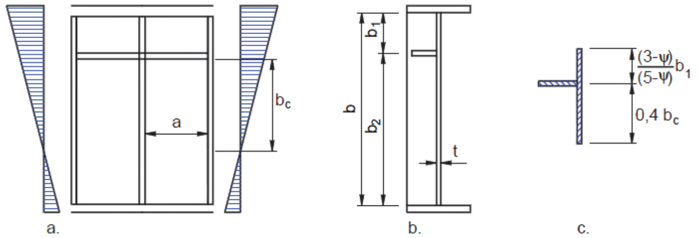
EN1993-1-5:2006 (A.4)

where,

$$a_c = 4.33\sqrt[4]{\frac{I_{sl,1}b_1^2 b_2^2}{t_3 b}} \qquad (2.12)$$

$A_{sl,1}$: *The gross area of the column.*
$I_{sl,1}$: *The second moment of area of the gross cross-section of the column.*
b_1, b_2 : *The distances from the longitudinal edges of the web to the stiffener.*

EN1993-1-5:2006
Figure A.2



[Fig. 2.13] Notations for a web plate with single stiffener in the compression zone

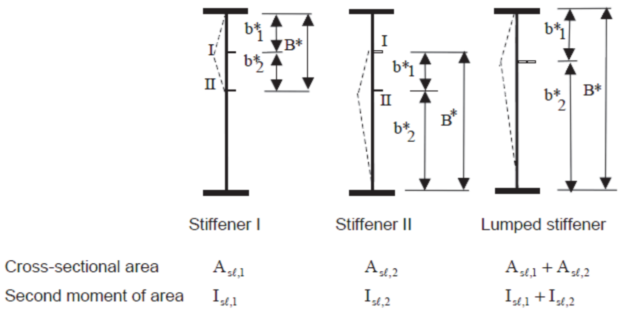
- with two stiffeners in the compression zone

  $\sigma_{cr,p}$ should be taken as the lowest of those computed for the 3 cases using equation (2.13) with $b_1=b_1^*$, $b_2=b_2^*$, $b=B^*$. The stiffeners in tension zone should be ignored.

$$\sigma_{cr,p} = \min\left[\sigma_{cr,sl,I}, \sigma_{cr,sl,II}, \sigma_{cr,sl,lumped}\right] \qquad (2.13)$$

EN1993-1-5:2006
A.2.1(7)

EN1993-1-5:2006
Figure A.3



[Fig. 2.14] Notations for plate with two stiffeners in the compression zone

It is assumed that one of stiffeners buckles while the other one acts as a rigid support.

Buckling of both the stiffeners simultaneously is accounted for by considering a single lumped stiffener that is substituted for both individual ones such that :

(a) its cross-sectional area and its second moment of area I_{st} are respectively the sum of for the individual stiffeners.

(b) it is positioned at the location of the resultant of the respective forces in the individual stiffeners.

- with at least three stiffeners in the compression zone

$$\sigma_{cr,p} = k_{\sigma,p}\sigma_E \qquad (2.14)$$

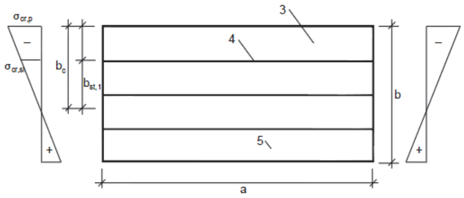
EN1993-1-5:2006
A.1(2)

where,

$$\sigma_E = \frac{\pi^2 E t^2}{12(1-\nu^2)b^2} \qquad (2.15)$$

$k_{\sigma,p}$: The buckling coefficient.
b is defined in Fig. 2.15.
t : The thickness of the plate.
E : The modulus of elasticity of structural steel.
v : The poisson's ratio

EN1993-1-5:2006 FigureA.1



[Fig. 2.15] Notations for longitudinally stiffened plates (1)

$k_{\sigma,p}$ may be approximated as shown in the following table.

EN1993-1-5:2006 (A.2)

[Table 2.11] Calculation of $k_{\sigma,p}$

Condition	$k_{\sigma,p}$
$\alpha \le \sqrt[4]{\gamma}$	$\dfrac{2\left(\left(1+\alpha^2\right)^2+\gamma-1\right)}{\alpha^2(\psi+1)(1+\delta)}$
$\alpha > \sqrt[4]{\gamma}$	$\dfrac{4\left(1+\sqrt{\lambda}\right)}{(\psi+1)(1+\delta)}$

where,

$$\psi = \frac{\sigma_2}{\sigma_1} \ge 0.5 \tag{2.16}$$

$$\gamma = \frac{\sum I_{sl}}{I_p} \tag{2.17}$$

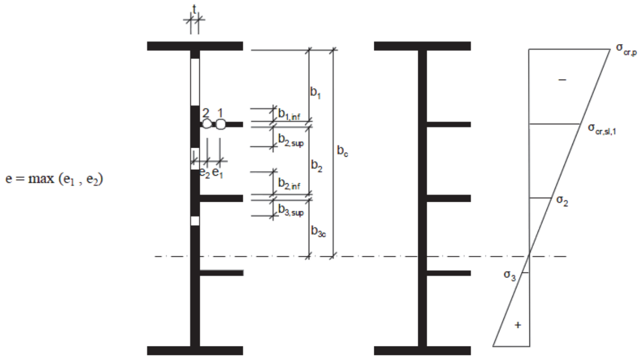
$$\delta = \frac{\sum A_{sl}}{A_p} \tag{2.18}$$

$$\alpha = \frac{a}{b} \ge 0.5 \tag{2.19}$$

$\sum I_{sl}$: The sum of the second moment of area of the whole stiffened plate.
$\sum A_{sl}$: The sum of the gross area of the individual longitudinal stiffener.
I_p : The second moment of area for bending of the plate.

$$I_p = \frac{bt^3}{12(1-\upsilon^2)} \tag{2.20}$$

A_p : The gross area of the plate = bt.
σ_1 : The larger edge stress.
σ_2 : The smaller edge stress.
a, b, t : As defined in Fig. 2.16.



	width for gross area	width for effective area according to Table 4.1	condition for ψ_i
$b_{1,inf}$	$\frac{3-\psi_1}{5-\psi_1} b_1$	$\frac{3-\psi_1}{5-\psi_1} b_{1,eff}$	$\psi_1 = \frac{\sigma_{cr,sl,1}}{\sigma_{cr,p}} > 0$
$b_{2,sup}$	$\frac{2}{5-\psi_2} b_2$	$\frac{2}{5-\psi_2} b_{2,eff}$	$\psi_2 = \frac{\sigma_2}{\sigma_{cr,sl,1}} > 0$
$b_{2,inf}$	$\frac{3-\psi_2}{5-\psi_2} b_2$	$\frac{3-\psi_2}{5-\psi_2} b_{2,eff}$	$\psi_2 > 0$
$b_{3,sup}$	$0,4\ b_{c3}$	$0,4\ b_{c3,eff}$	$\psi_3 = \frac{\sigma_3}{\sigma_2} < 0$

[Fig. 2.16] Notations for longitudinally stiffened plates (2)

EN1993-1-5:2006
Figure A.1

(3) Plate type behavior.

• The relative plate slenderness $\overline{\lambda}_p$ of the equivalent plate

$$\overline{\lambda}_p = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,p}}} \qquad (2.21)$$

EN1993-1-5:2006
4.5.2(1)

where,

$$\beta_{A,c} = \frac{A_{c,eff,loc}}{A_c} \qquad (2.22)$$

A_c : *The gross area of the compression zone of the stiffened plate except the parts of subpanels supported by an adjacent plate.*

$A_{c,eff,loc}$: *The effective area of the same part of the plate with due allowance made for possible plate buckling of subpanels and/or of stiffened panels.*

• The reduction factor ρ

[Table 2.12] Calculation of ρ

Element	Condition	ρ
Internal element	$\overline{\lambda}_p \le 0.673$	1.0
	$\overline{\lambda}_p > 0.673$ where, $(3+\psi) \ge 0$	$\frac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^2} \le 1.0$
Outstand element	$\overline{\lambda}_p \le 0.748$	1.0
	$\overline{\lambda}_p > 0.748$	$\frac{\overline{\lambda}_p - 0.188}{\overline{\lambda}_p^2} \le 1.0$

EN1993-1-5:2006
4.4(2)

(4) Column type behavior.

- The elastic critical column buckling stress $\sigma_{cr,c}$

(a) Unstiffened plate : $$\sigma_{cr,c} = \frac{\pi^2 Et^2}{12(1-\nu^2)a^2} \quad (2.23)$$

EN1993-1-5:2006 4.5.3(2),(3)

(b) Stiffened plate : $$\sigma_{cr,c} = \sigma_{cr,sl}\frac{b_c}{b_{sl,1}} \quad (2.24)$$

where,
a : Length of a stiffened or unstiffened plate.

$$\sigma_{cr,sl} = \frac{\pi^2 EI_{sl,1}}{A_{sl,1}a^2} \quad (2.25)$$

$I_{sl,1}$: The second moment of area of the stiffener, relative to out-of-plane bending of the plate.
$A_{sl,1}$: The gross cross-sectional area of the stiffener and the adjacent parts of the plate.

- The relative column slenderness $\bar{\lambda}_c$

(a) Unstiffened plate : $$\bar{\lambda}_p > 0.673 \quad (2.26)$$

(b) Stiffened plate : $$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,c}}} \quad (2.27)$$

EN1993-1-5:2006 4.5.3(4)

where,

$$\beta_{A,c} = \frac{A_{sl,1,eff}}{A_{sl,1}} \quad (2.28)$$

$A_{sl,1,eff}$: The effective cross-sectional area of the stiffener with due allowance for plate buckling.

- The reduction factor χ_c

$$\chi_c = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_c^2}} \le 1.0 \quad (2.29)$$

EN1993-1-1:2005 6.3.1.2

(a) Unstiffened plate : $$\Phi = 0.5\left[1 + \alpha(\bar{\lambda}_c - 0.2) + \bar{\lambda}_c^2\right] \quad (2.30)$$

where, α = 0.21

(b) Stiffened plate : $$\Phi = 0.5\left[1 + \alpha_e(\bar{\lambda}_c - 0.2) + \bar{\lambda}_c^2\right] \quad (2.31)$$

where,

$$\alpha_e = \alpha + \frac{0.09}{i/e} \quad (2.32)$$

EN1993-1-5:2006 4.5.3(5)

$$i = \sqrt{\frac{I_{sl,1}}{A_{sl,1}}} \quad (2.33)$$

e = max(e1, e2) is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the column.
α = 0.34 (for closed section stiffener), 0.49 (for open section stiffener)

(5) Final reduction factor ρ_c from interaction between plate and column buckling.

EN1993-1-5:2006 4.5.4(1)

$$\rho_c = (\rho - \chi_c)\xi(2-\xi) + \chi_c \tag{2.34}$$

where,

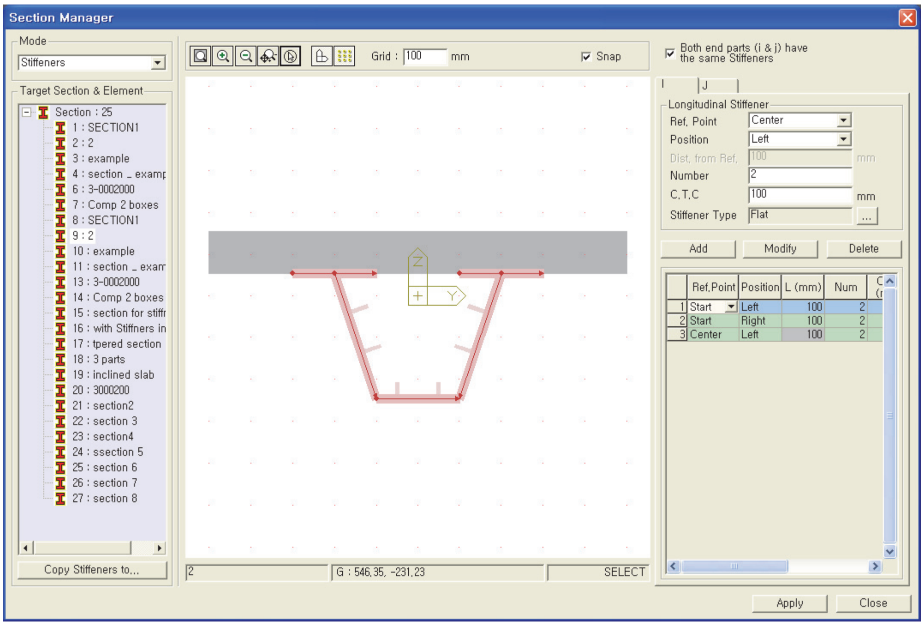
$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1, \quad 0 \le \xi \le 1.0 \tag{2.35}$$

$\sigma_{cr,p}$: *The elastic critical plate buckling stress.*
$\sigma_{cr,c}$: *The elastic critical column buckling stress.*
χ_c : *The reduction factor due to column buckling.*

Longitudinal stiffener

Longitudinal stiffeners of box girder need to be entered by section properties. Flat, Tee, U-Rib type stiffener can be defined.

Design>Composite Steel Girder Design>Longitudinal Stiffener...



[Fig. 2.17] Section Manager, Longitudinal stiffener Dialog

1.8 Calculate bending resistance, M_{Rd}

Bending resistance, M_{Rd}, can be calculated as follows based on its class.

Class 1 or 2 cross-sections can be checked by using the plastic or elastic bending resistance.

Class 3 cross-sections are checked with the elastic bending resistance, or possibly reclassified as effective Class 2 cross-section and then checked with the plastic bending resistance.

Class 4 cross-sections are also checked with the elastic bending resistance but by using the effective cross-section, reduced to take account of buckling.

(1) Class 1 and 2 + Positive Moment.

• The strength of the reinforcing steel bars in compression is neglected.

• General case : $M_{Rd} = M_{pl,Rd}$ (2.36)

• For the structural steel grade S420 or S460, M_{Rd} is calculated as shown in the table below.

[Table 2.13] Calculation M_{Rd}

EN1994-2:2005 6.2.1.4(6)

Condition		M_{Rd}
$x_{pl} \le 0.15h$		$M_{pl,Rd}$
$0.15h < x_{pl} \le 0.4h$		$\beta M_{pl,Rd}$
$x_{pl} > 0.4h$	$N_c \le N_{c,el}$	$M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\dfrac{N_c}{N_{c,el}}$
	$N_{c,el} < N_c \le N_{c,f}$	$M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Ed}\right)\dfrac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}}$

where,

$M_{pl,Rd}$: *Design value of the plastic resistance moment of the composite section with full shear connection.*

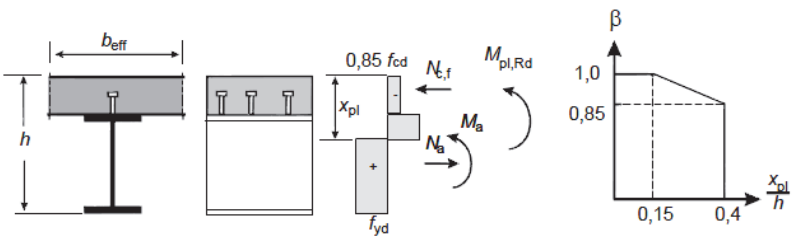
$M_{el,Rd}$: *Design value of the elastic resistance moment of the composite section.*

β : *The reduction factor.*

N_c : *Design value of the compressive normal force in the concrete flange.*

$N_{c,el}$: *Compressive normal force in the concrete flange corresponding to $M_{el,Rd}$.*

$N_{c,f}$: *Design value of the compressive normal force in the concrete flange with full shear connection.*



[Fig. 2.18] Reduction factor β for $M_{pl,Rd}$

(2) Class 1 and 2 + Negative Moment.

• The strength of the concrete in tension is neglected.

• Bending resistance

$$M_{Rd} = M_{pl,Rd} \tag{2.37}$$

(3) Class 3

• Bending resistance

$$M_{Rd} = M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \tag{2.38}$$

EN1994-2:2005 (6.4)

(4) Class 4

• Section properties should be calculated by considering the effective area. If the section is under the compression, the additional moment must be taken in to account due to the eccentricity between the gravity center of gross section and effective section.

Refer to the clause 1.5 to see how to calculate the effective area and additional moment.

• Bending resistance

$$M_{Rd} = M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \tag{2.39}$$

EN1994-2:2005 (6.4)

1.9 Check bending resistance

$$M_{Ed} \leq M_{Rd} \qquad (2.40)$$

where,
M_{Ed} *: Design bending moment.*
M_{Rd} *: Design moment resistance.*

• Load combination

In midas Civil, bending resistance will be verified for the load combinations that the Active column is specified as Strength/Stress in Results>Load combinations>Steel Design tab.

1.10 Verification of Bending Resistance

By Result Table

Bending resistance can be verified in the table format as shown below.

Design>Composite Steel Girder Design>Design Result Tables>Bending Resistance…

Model View / Bending Resistance

Elem	Position	Positive/Negative	Lcom	Type	Top Class	Bot Class	Web Class	Sect. Class	Ma,Ed (kN·m)	Mc,Ed (kN·m)	Mpl,Rd (kN·m)	Mel,Rd (kN·m)	M_Rd (kN·m)
2	I[2]	Negative	sLCB1	-	1	2	1	2	-36.4051	-1240.8507	14706.4069	7821.1822	14706.4069
2	I[2]	Positive	-	-	-	-	-	-	-	-	-	-	-
2	J[3]	Negative	-	-	-	-	-	-	-	-	-	-	-
2	J[3]	Positive	-	-	-	-	-	-	-	-	-	-	-

Elem: Element
Position: I/J-end
Positive/Negative: Positive/Negative moment
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
Top Class: Class of top flange
Bot Class: Class of bottom flange
Web Class: Class of web
Sect. Class: Class of cross section
Ma,Ed: The design bending moment applied to structural steel section before composite behavior
Mc,Ed: The part of the design bending moment acting on the composite section
Mpl,Rd: Design value of the plastic resistance moment of the composite section
Mel,Rd: Design value of the elastic resistance moment of the composite section
M_Rd: Design value of the resistance moment of a composite section

By Excel Report

Detail results with applied equations and parameters can be checked in the Excel Report.

2 **Bending Resistance**

2.1 Negative Moment

- Design load

Load combination name : sLCB1

$N_{a,Ed}$ = 0.000 kN

$N_{c,Ed}$ = -2000.000 kN

$M_{a,Ed}$ = -36.405 kN · m

$M_{c,Ed}$ = -1240.851 kN · m

- Stress

Top Flange

Left	y_1	-200.000 mm	z_1	607.710 mm	σ_1	23.487 MPa
	y_2	-15.000 mm	z_2	607.710 mm	σ_2	23.487 MPa
Right	y_1	200.000 mm	z_1	607.710 mm	σ_1	23.487 MPa
	y_2	15.000 mm	z_2	607.710 mm	σ_2	23.487 MPa

- Classification of sections

Part	Class
Top flange	1
Web	1
Bottom flange	2
Section	2

- Plastic resistance moment, $M_{pl,Rd}$

Plastic NA = 613.574 mm

N_{slab} = 0.000 kN

N_{rebar} = 2895.429 kN

$N_{g,top}$ = 14158.536 kN (Upper side of PNA)

$N_{g,bot}$ = 17053.964 kN (Lower side of PNA)

$M_{pl,Rd}$ = 14706.407 kN · m

M_{Rd} = $M_{pl,Rd}$ = 14706.407 kN · m

M_{Rd} = 14706.407 kN · m > M_{Ed} = -1277.256 kN · m ...OK

2. Resistance to vertical shear

Limit state of vertical shear resistance will satisfy the condition, $V_{Ed} \leq V_{Rd}$.

Shear resistance, V_{Rd} , will be determined as smaller value between $V_{pl,Rd}$ and $V_{b,Rd}$ when considering shear buckling. When the shear buckling is not considered, Shear resistance, V_{Rd} , will be determined as $V_{pl,Rd}$. The plastic resistance and buckling resistance are calculated as follows.

2.1 Plastic resistance to vertical shear

$$V_{pl,Rd} = V_{pl,a,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} \tag{2.41}$$

EN1994-2:2005 6.2.2.2
EN1993-1-1:2005 (6.18)

where,
γ_{M0} *: The partial factor for resistance of cross-sections whatever the class is.*
A_v *: The shear area. In midas Civil, only welded I, H and box sections are considered.*

$$A_v = \eta \sum (h_w t_w) \tag{2.42}$$

EN1993-1-1:2005 6.2.6(3)-d)

h_w *: The depth of the web*
t_w *: The web thickness*
η *: The coefficient that includes the increase of shear resistance at web slenderness*

[Table 2.14] Coefficient η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

2.2 Shear buckling resistance

Plates with $\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$ for an unstiffened web, or $\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$ for a stiffened web, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports.

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{2.43}$$

EN1994-2:2005 6.2.2.3
EN1993-1-5:2006 (5.1)

(1) Contribution from the web $V_{bw,Rd}$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{2.44}$$

EN1993-1-5:2006 (5.2)

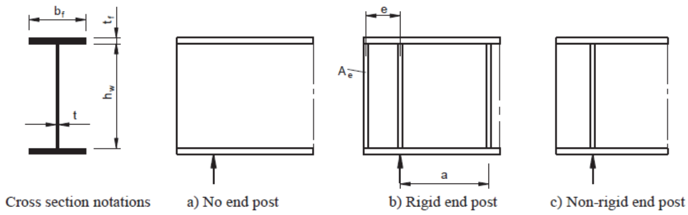
where,
f_{yw} *: Yield strength of the web.*
h_w *: Clear web depth between flanges.*
t *: Thickness of the plate.*
γ_{M1} *: Partial factor for resistance of members to instability assessed by member checks.*
χ_w *: Factor for the contribution of the web to the shear buckling resistance.*

[Table 2.15] Contribution from the web χw

EN1993-1-5:2006 Table 5.1

Condition	Rigid end post	Non-rigid end post
$\overline{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \le \overline{\lambda}_w < 1.08$	$0.83/\overline{\lambda}_w$	$0.83/\overline{\lambda}_w$
$\overline{\lambda}_w \ge 1.08$	$1.37/(0.7+\overline{\lambda}_w)$	$0.83/\overline{\lambda}_w$

EN1993-1-5:2006 Figure 5.1



[Fig. 2.19] End supports

λw : Slenderness parameter.

[Table 2.16] Calculation of λ_w

EN1993-1-5:2006 5.3(3)

Condition	$\overline{\lambda}_w$
Transverse stiffeners at supports only. (In midas Civil, when longitudinal stiffener exists only)	$\overline{\lambda}_w = \dfrac{h_w}{86.4t\varepsilon}$
Transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both (In midas Civil, except for the condition when longitudinal stiffener exists only)	$\overline{\lambda}_w = \dfrac{h_w}{37.4t\varepsilon\sqrt{k_\tau}}$

For webs with longitudinal stiffeners,

$$\overline{\lambda}_w \ge \frac{h_{wi}}{37.4t\varepsilon\sqrt{k_{\tau i}}} \qquad (2.45)$$

EN1993-1-5:2006 5.3(5)

h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter λ_w of all subpanels within the web panel under consideration. ($k_{\tau st} = 0$)

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (2.46)$$

k_τ : The **minimum** *shear buckling coefficient for the web panel.*

[Table 2.17] Calculation kτ

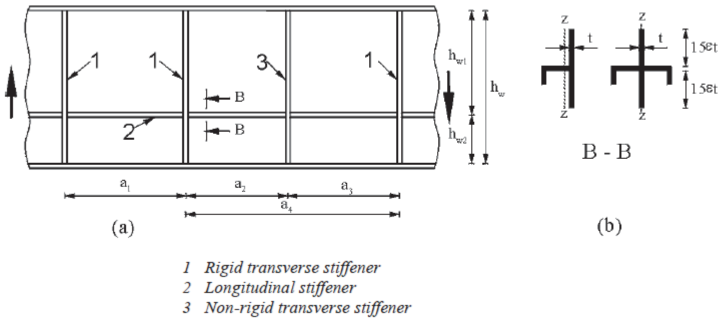
EN1993-1-5:2006 A.3

No. of longitudinal stiffeners	Condition	k_τ
= 0 or >2	$a/h_w \ge 1.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$a/h_w < 1.0$	$k_\tau = 4.00 + 5.34(h_w/a)^2 + k_{\tau st}$
1 or 2	$\alpha = a/h_w \ge 3.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$\alpha = a/h_w < 3.0$	$k_\tau = 4.1 + \dfrac{6.3 + 0.18\dfrac{I_{sl}}{t^3 h_w}}{\alpha^2} + 2.2\sqrt[3]{\dfrac{I_{sl}}{t^3 h_w}}$

$$k_{\tau st} = 9\left(\frac{h_w}{a}\right)^2 \sqrt[4]{\left(\frac{I_{sl}}{t^3 h_w}\right)^3} \geq \frac{2.1}{t}\sqrt[3]{\frac{I_{sl}}{h_w}} \tag{2.47}$$

a : The distance between transverse stiffeners.

EN1993-1-5:2006
Figure 5.3



[Fig. 2.20] Web with transverse and longitudinal stiffeners

I_{sl} : The second moment of area of the longitudinal stiffener about z-axis. The value of I_{sl} will be multiplied by 1/3 when calculating k_τ.
η : The coefficient that includes the increase of shear resistance at web slenderness

[Table 2.18] Calculation η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

(2) Calculation of the shear stress in the flange $\tau_{Ed,max}$

• Structural steel box section

$$\tau_{Ed,a} = \frac{V_{Ed,a}}{I_a}\frac{Q_{f,a}}{t_f} \tag{2.48}$$

• Composite box section

$$\tau_{Ed,c} = \frac{V_{Ed,c}}{I_c}\frac{Q_{f,c}}{t_f} \tag{2.49}$$

$$\tau_{Ed,\max} = \tau_{Ed,a} + \tau_{Ed,c} \tag{2.50}$$

$$\tau_{Rd} = \frac{\chi f_{yf}}{\sqrt{3}\gamma_{M1}} \tag{2.51}$$

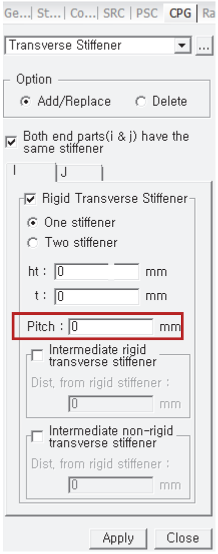
where,
$Q_{f,a}$: Geometric moment of area in flange before composite
$Q_{f,c}$: Geometric moment of area in flange after composite
I_a : Second moment of area in flange before composite
I_c : Second moment of area in flange after composite
f_{yf} : Yield strength of the flange.
$V_{Ed,a}$: Shear force of girder before composite
$V_{Ed,c}$: Shear force of girder after composite
γ_{M1} : Partial factor for resistance of members to instability assessed by member checks.
χ : Apply the value of "1.2".

In case of box girder, shear resistance verification in flange will be done by comparing the maximum shear force, $\tau_{Ed,max}$, to the shear resistance, τ_{Rd}.

Transverse stiffener

Transverse stiffeners can be specified by members.

Design>Composite Steel Girder Design>Transverse Stiffener...



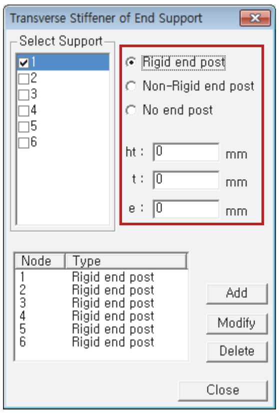
a: Spacing of rigid transverse stiffeners

[Fig. 2.21] Transverse stiffener

Transverse stiffener of end support

Transverse stiffener of end support can be entered from the following dialog box. End support type by nodes and related parameter can be defined.

Design>Composite Steel Girder Design>Transverse Stiffener of End Support...



Type of end support (See Fig. 2.17)

[Fig. 2.22] Transverse Stiffener of End Support

2.3 Resistance to vertical shear

V_{Rd} is calculated depending on the value of h_w/t as shown in the table below.

[Table 2. 19] Calculation of V_{Rd}

	Condition	V_{Rd}
Unstiffened	$\frac{h_w}{t} \le \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{b,Rd}$
Stiffened	$\frac{h_w}{t} \le \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{b,Rd}$

where,
$V_{pl,Rd}$ *: The plastic resistance to vertical shear.*
$V_{b,Rd}$ *: The shear buckling resistance.*

2.4 Interaction bending and vertical shear

(1) Verification condition of interaction between sear force and bending moment

When the following condition is satisfied, combined effects of bending and shear need to be verified.

EN1994-2:2005 6.2.2.4(1)

$$\overline{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} > 0.5 \tag{2.52}$$

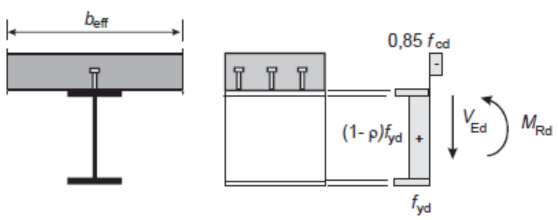
where,
V_{Ed} *: The design shear force including shear from torque.*
$V_{bw,Rd}$ *: The design resistance for shear of contribution from the web.*

(2) For cross-sections in Class1 or 2

Apply the reduced design steel strength $(1\text{-}\rho)f_{yd}$ in the shear area. It is not considered in midas Civil.

$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2 \tag{2.53}$$

EN1994-2:2005 6.2.2.4(2) Figure 6.7



[Fig. 2.23] Plastic stress distribution modified by the effect of vertical shear

(3) For cross-sections in Class3 and 4

- $\bar{\eta}_3 \leq 0.5$: M_{Rd}, N_{Rd} need not be reduced.
- $\bar{\eta}_3 > 0.5$: The combined effects of bending and shear in the web of an I or box girder should satisfy.

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \leq 1.0 \qquad (2.54)$$

EN1993-1-5:2006 7.1(1)

where,

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}} \geq \frac{M_{f,Rd}}{M_{pl,Rd}} \qquad (2.55)$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \qquad (2.56)$$

2.5 Check resistance to vertical shear

$$V_{Ed} \leq V_{Rd} \qquad (2.57)$$

where,
V_{Ed} *: Design value of the shear force acting on the composite section.*
V_{Rd} *: Design value of the resistance of the composite section to vertical shear.*

2.6 Verification of vertical shear resistance

By Result Table

The verification results can be checked in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Vertical Shear…

Model View | Resistance to Vertical Shear

Elem	Position	Lcom	Type	Top Class	Bot Class	Web Class	Sect. Class	N_Ed (kN)	M_Ed (kN·m)	V_Ed (kN)	Vpl,Rd (kN)	Vb,Rd (kN)
2	I[2]	sLCB1	-	1	1	2	2	-2000.0000	3184.2313	-46985.8208	48560.9168	0.0000
2	J[3]	-	-	-	-	-	-	-	-	-	-	-

Position: I/J-end
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
Top Class: Class of top flange
Bot Class: Class of bottom flange
Web Class: Class of web
Sect. Class: Class of cross section
N_Ed : Design value of the compressive normal force
M_Ed: Design bending moment
V_Ed : Design value of the shear force acting on the composite section
Vpl,Rd: Design value of the plastic resistance of the composite section to vertical shear
Vb,Rd: Design value of the shear buckling resistance of a steel web

By Excel Report

Detail results with applied equations and parameters can be checked in the Excel Report.

93	3 **Resistance to Vertical Shear**							
94	- Design load							
95	Load combination name :		sLCB1					
96	N_{Ed}	=	-2000.000	kN				
97	$M_{a,Ed}$	=	-804.929	kN · m				
98	$M_{c,Ed}$	=	-1848.218	kN · m				
99	$V_{Ed,a}$	=	-14488.720	kN				
100	$V_{Ed,c}$	=	-32497.101	kN				
101	V_{Ed}	=	-46985.821	kN				
102								
136	- Plastic resistance moment, $M_{pl,Rd}$							
137	Plastic NA	=	1343.878	mm				
138								
139	N_{slab}	=	0.000	kN				
140	$N_{g,top}$	=	73162.780	kN				
141	$N_{g,bot}$	=	81270.695	kN				
142								
143	$M_{pl,Rd}$	=	115704.166	kN · m				
144								
145	- Calculation. $V_{pl,Rd}$							
146	Web(Web_R)							
147	α	=	a/h_w		=	0.14607076		
148	k_τ	=	$4.1+(6.3+0.18 \cdot I_{sl}/(t^3 \cdot h_w)) / \alpha^2 + 2.2 \cdot (I_{sl}/(t^3 \cdot h_w))^{1/3}$		=	306.7841206		
149	I_{sl}	=	167648996.696	mm^4				
150	t	=	50.000	mm				
151								
152	$V_{pl,Rd}$	=	$Av \cdot (fy / \sqrt{(3)}) / \gamma_{M0}$		=	24308.110	kN	
153	V_{Rd}	=	24308.110	kN				
154	V_{Edi}	=	V_{Ed} / Num. of Web		=	-23492.910	kN	
155								
156	V_{Edi} / V_{Rd}	=	0.966		≤	1.0		... OK
157								
158	Interaction M-V							
159	For the section class 1 or 2, M-V interaction should be checked separately by the user.							

3. Resistance to longitudinal shear

Resistance to longitudinal is verified only for the plate I-girder and the following condition must be satisfied.

$$v_{L,Ed} \le v_{L,Rd}$$

$v_{L,Ed}$, $v_{L,Rd}$ shall be calculated as follows.

3.1 Design shear resistance of headed stud

EN1994-2:2005 6.6.3.1(1)

$$P_{Rd} = \min\left[P_{Rd1}, P_{Rd2}\right] \quad (2.58)$$

$$P_{Rd1} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} \quad (2.59)$$

$$P_{Rd2} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \quad (2.60)$$

where,
γ_V : *The partial factor.*
d : *The diameter of the shank of the stud.*
f_u : *The specified ultimate tensile strength of the material of the stud.*
f_{ck} : *The characteristic cylinder compressive strength of the concrete at the age considered.*
h_{sc} : *The overall nominal height of the stud.*

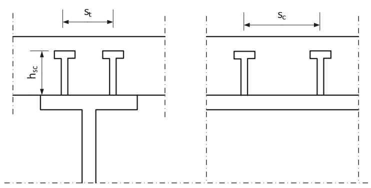
EN1994-2:2005 (6.20),(6.21)

[Table 2.20] Calculation of α

$3 \le h_{sc}/d \le 4$	$h_{sc}/d > 4$
$\alpha = 0.2\left(\frac{h_{sc}}{d} + 1\right)$	α =1

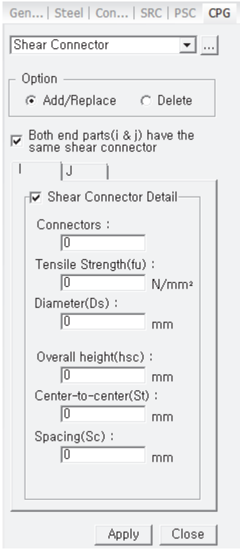
Shear connector

For shear connectors, enter the number of connectors, tensile strength, dimension, height (h_{sc}), transverse spacing (s_t), and longitudinal spacing (s_c).



[Fig. 2.24] Notation of shear connector

Design>Composite Steel Girder Design>Shear Connector...



[Fig. 2.25] Shear connector Input Dialog

3.2 Bearing shear stress of shear connector, $v_{L,Rd}$

$$v_{L,Rd} = \frac{P_{Rd} N}{s_c} \tag{2.61}$$

where,

N : The number of the shear connector.

s_c : The space of the shear connector.

3.3 Shear stress at the connection between girder and deck, $v_{L,Ed}$

(1) Beams with cross-sections in Class 1 or 2 and under the sagging moment and inelastic behavior ($M_{Ed} > M_{el,Rd}$)

$$v_{L,Ed} = \frac{V_{L,Ed}}{L_v} \tag{2.62}$$

where,

$$V_{L,Ed} = \frac{\left(N_{c,f} - N_{c,el}\right)\left(M_{ED} - M_{el,Rd}\right)}{M_{pl,Rd} - M_{el,Rd}} \tag{2.63}$$

EN 1994-2: 2005
6.6.2.2

L_v : Length of shear connection. ($L_v = b_{eff} = B_c$)

(2) Other cases

$$v_{L,Ed} = \frac{V_{Ed} Q_s}{I_y} \tag{2.64}$$

where,

Q_s : Geometric moment of area at the shear connector position (contact point between girder and slab)

[Table 2.21] Calculation of Q_s

Condition	Q_s
Gravity center of composite section < Height of girder	Calculate the geometric moment of area with slab
Gravity center of composite section ≥ Height of girder	Calculate the geometric moment of area with girder

3.4 Check resistance to longitudinal shear

$$v_{L,Ed} \leq v_{L,Rd} \tag{2.65}$$

where,

$v_{L,Ed}$: *Design longitudinal shear force per unit length at the interface between steel and concrete.*

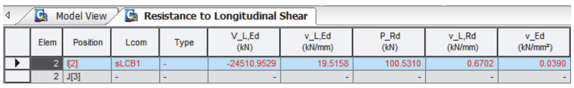
$v_{L,Rd}$: *Resistance to longitudinal shear.*

3.5 Verification of longitudinal shear resistance

By Result Table

Verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Longitudinal Shear...



Model View | Resistance to Longitudinal Shear

	Elem	Position	Lcom	Type	V_L,Ed (kN)	v_L,Ed (kN/mm)	P_Rd (kN)	v_L,Rd (kN/mm)	v_Ed (kN/mm²)
▶	2	I[2]	sLCB1	-	-24510.9529	19.5158	100.5310	0.6702	0.0390
	2	J[3]	-	-	-	-	-	-	-

Elem: Element

Position: I/J-end

Lcom: Load combination

Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)

V_L,Ed: Longitudinal shear force acting on length of the inelastic region

v_L,Ed: Design longitudinal shear force per unit length at the interface between steel and concrete

P_Rd: Design value of the shear resistance of a single connector

v_L,Rd:

v_Ed: Design longitudinal shear stress

By Excel Report

6 Resistance to Longitudinal Shear

- Design load

Load combination name : sLCB1

$N_{c,el}$	=	0.000	kN
$N_{c,f}$	=	0.000	kN
M_{Ed}	=	-1234.865	kN · m
V_{Ed}	=	-24510.953	kN
$M_{pl,Rd}$	=	13217.334	kN · m
$M_{el,Rd}$	=	8780.518	kN · m

- Shear resistance of a single connector

$P_{Rd,1}$	=	$0.8 \cdot f_u \cdot \pi \cdot d^2 / 4 / \gamma_V$	=	100.531	kN
$P_{Rd,2}$	=	$0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{(f_{ck} \cdot E_{cm})} / \gamma_V$	=	137.253	kN
P_{Rd}	=	$Min(P_{Rd,1}\ ,\ P_{Rd,2})$	=	100.531	kN

where, f_u	=	400.000	MPa	
α	=	1		for $h_{sc}/d > 4$
Num.	=	2		
d	=	20.000	mm	
h_{sc}	=	100.000	mm	
Space	=	300.000	mm	

- Verification

$v_{L,Ed}$	=	$V_{Ed} \cdot (A \cdot z / I)$	=	19515.797	kN/m
$v_{L,Rd}$	=	$P_{Rd} \cdot$ Num./Space	=	670.206	kN/m
$v_{L,Ed}$	>	$v_{L,Rd}$	...	NG	

Chapter 2. Composite Steel Box Girder Design: EN1994-2

Serviceability Limit States

1. Stress limitation

For the stress limit check of box girder, the following stress will be calculated and compared to its allowable stress: Normal stress of girders, Shear stress of girders, Combined stress of girders, stress in slab, and stress in rebar. Each stress can be calculated as follows.

1.1 Stress limitation for girder

(1) Normal stress $\sigma_{Ed,ser}$

$$\sigma_{Ed,ser} \le \sigma_{allow} = \frac{f_y}{\gamma_{M,ser}} \tag{2.66}$$

• Stress in girder, $\sigma_{Ed,ser}$, is calculated by the stresses summation of before composite and after composite state at 4 different points. Member forces and section properties are calculated as shown in the table below.

[Table 2.22] Member forces for calculating girder stress

Type	Before composite	After composite
Section Properties	Girder	Sagging moment : Deck concrete + Girder
Member Force	Calculate using girder only	Calculate considering deck concrete and girder

In midas Civil, applied section properties can be verified in the excel report. The section properties of before composite action is shown as “Before”, after composite action is shown as “After”, negative moment with considering cracked section is shown as “Crack”.

(2) Shear stress $\tau_{Ed,ser}$

$$\tau_{Ed,ser} \le \tau_{allow} = \frac{f_y}{\sqrt{3}\gamma_{M,ser}} \tag{2.67}$$

EN1993-2:2006 (7.2)

where,

$$\tau_{Ed,ser} = \frac{V_{Ed}}{A_v} \tag{2.68}$$

V_{Ed} : *Shear force after composite action*
A_v : *Shear area. For I-girder,* $A_v = h_w t_w$. *For the other sections,* $A_v = \sum A_{web}$.

(3) Combined stress $\sigma_{Ed,com,ser}$

EN1994-2:2005 7.2.2(5)

EN1993-2:2006 (7.1)

$$\sigma_{Ed,com,ser} \le \sigma_{allow} = \frac{f_y}{\gamma_{M,ser}} \tag{2.69}$$

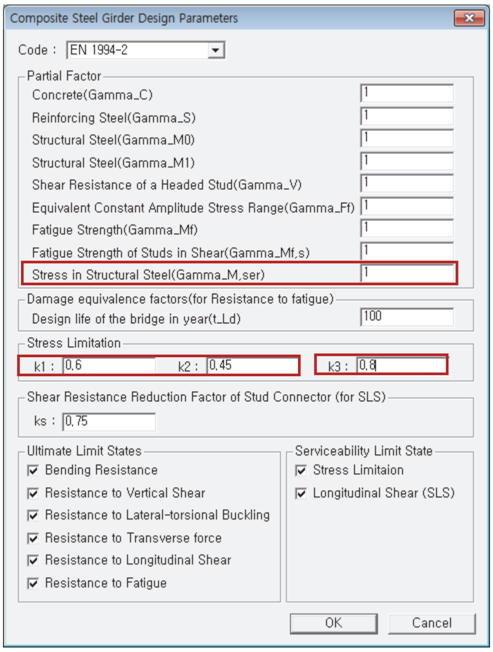
where,

EN1993-2:2006 (7.3)

$$\sigma_{Ed,com,ser} = \sqrt{\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2} \tag{2.70}$$

Stress limitation parameters

Design > Composite Steel Girder Design > Design Parameters...



$\gamma_{M,ser}$

k_1, k_2, k_3

[Fig. 2.26] Composite Girder Design Parameters

1.2 Stress limitation for concrete of slab

$$\sigma_c \le \sigma_{allow} = kf_{ck} \tag{2.71}$$

where,
k : It is used as the user defined value.

EN1994-2:2005 7.2.2(2)

[Table 2.23] Recommended value of k for concrete

Serviceability Load combination Type	k	
	Applied	Recommended
Characteristic	k_1	0.6
Quasi-permanent	k_2	0.45

f_{ck} : The characteristic value of the cylinder compressive strength of concrete at 28 days.

1.3 Stress limitation for reinforcement of slab

$$\sigma_s \leq \sigma_{allow} = k_3 f_{sk} \quad (2.72)$$

where,

k_3 : It is used as the user defined value.

EN1994-2:2005 7.2.2(4

[Table 2.24] Recommended value of k for reinforcement

Serviceability Load combination Type	k	
	Applied	Recommended
Characteristic	k_3	0.45

f_{sk} : Characteristic value of the yield strength of reinforcing steel.

1.4 Verification of stress limitation resistance

By Result Table

The verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Stress Limitation···

Model View | Stress Limitation

Elem	Position	Top and Bottom Flange of Structural Steel								Concrete Deck				Reinforcement in Deck			
		Lcom	Type	Sigma_Ed,ser (N/mm²)	ALW (N/mm²)	Tau_Ed,ser (N/mm²)	ALW (N/mm²)	SQRT(sigma^2+3tau^2) (N/mm²)	ALW (N/mm²)	Lcom	Type	Sigma_c (N/mm²)	k*fck (N/mm²)	Lcom	Type	Sigma_s (N/mm²)	k*fsk (N/mm²)
2	I[2]	sLCB2	Characteris	30.0868	440.0000	158.4094	254.0341	276.0178	440.0000	sLCB2	Characteris	-0.0000	24.0000	sLCB2	Characteri	6.5638	320.0000
2	J[3]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Sigma_Ed,ser, Tau_Ed,ser: Nominal stresses in the structural steel from the characteristic load combination. Refer to EN 1993-2 7.3.

ALW: Stress limit.

Sigma_c: Stress in the concrete deck.

*k*fck: Stress limit.*

Sigma_s: stress in the reinforcement.

*k*fsk: stress limit.*

By Excel Report

A B C D E F G H I J K L M N O P Q R S T U V W X Y Z AA AB AC AD AE AF AG

8 **Stress Limitation**

- In the structural steel

Characteristic load combination name : sLCB3

$\sigma_{Ed,ser}$ = 65.343 MPa (Bottom-right fiber in the flange)

$\tau_{Ed,ser}$ = 612.899 MPa (Neutral axis in the web)

$\sigma_{Ed,ser}$ ≤ $f_y / \gamma_{M,ser}$

65.343 MPa ≤ 440.000 MPa ... OK

$\tau_{Ed,ser}$ ≤ $f_y / (\sqrt{3} \cdot \gamma_{M,ser})$

612.899 MPa > 254.034 MPa ... NG

$\sqrt{(\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2)}$ ≤ $f_y / \gamma_{M,ser}$

1063.582 MPa > 440.000 MPa ... NG

- In the concrete of the slab

Quasi-permanent load combination name : sLCB2

$\sigma_c \leq k_2 f_{ck}$

0.000 MPa ≤ 18.000 MPa ... OK

- In the reinforcement

Load combination name : sLCB3

$\sigma_s \leq k_3 f_{sk}$

19.995 MPa ≤ 320.000 MPa ... OK

2. Longitudinal shear in SLS (Serviceability Limit States)

Resistance to longitudinal shear can be verified for the I-girder and following condition must be satisfied.

$$v_{L,Ed} \le v_{L,Rd}$$

$v_{L,Ed}$, $v_{L,Rd}$ shall be calculated as follows.

2.1 Design shear resistance of headed stud

$$P_{Rd} = \min[P_{Rd1}, P_{Rd2}] \quad (2.73)$$

EN1994-2:2005 6.6.3.1(1)

$$P_{Rd1} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} \quad (2.74)$$

$$P_{Rd2} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \quad (2.75)$$

where,
γ_V : *The partial factor.*
d : *The diameter of the shank of the stud,* $16mm \le d \le 25mm$.
f_u : *The specified ultimate tensile strength of the material of the stud,* $\le 500N/mm^2$.
f_{ck} : *The characteristic cylinder compressive strength of the concrete at the age considered.*
h_{sc} : *The overall nominal height of the stud.*
α : *Refer to Table 2.20.*

EN1994-2:2005 (6.20),(6.21)

Shear connector parameters

Shear connector is entered by members. Refer to the clause 3.1 for the input method.

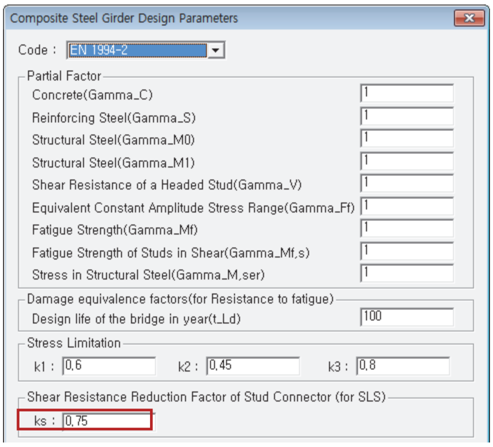
2.2 Bearing shear stress of shear connector, $v_{L,Rd}$

$$v_{L,Rd} = \frac{k_s P_{Rd} N_{conn}}{s_{conn}} \quad (2.76)$$

where,
k_s : *Reduction factor for shear resistance of stud connector.*
N_{conn} : *The number of the shear connector.*
s_{conn} : *The space of the shear connector.*

Reduction factor k_s

Reduction factor for stud, k_s, can be entered in Composite Steel Girder Design Parameters dialog box.



[Fig. 2.27] Composite Girder Design Parameters

2.3 Shear stress at the connection between girder and deck, $v_{L,Ed}$

(1) Beams with cross-sections in Class 1 or 2 and under the sagging moment and inelastic behavior ($M_{Ed} > M_{el,Rd}$).

$$v_{L,Ed} = \frac{V_{L,Ed}}{L_v} \tag{2.77}$$

where,

$$V_{L,Ed} = \frac{\left(N_{c,f} - N_{c,el}\right)\left(M_{ED} - M_{el,Rd}\right)}{M_{pl,Rd} - M_{el,Rd}} \tag{2.78}$$

EN 1994-2: 2005
6.6.2.2

L_v : Length of shear connection. ($L_v = b_{eff} = B_c$)

(2) Other cases

$$v_{L,Ed} = \frac{V_{Ed} Q_s}{I_y} \tag{2.79}$$

where,

Q_s : Geometric moment of area at the shear connector position (contact point between girder and slab). Refer to Table 2.21 to see the calculation method.

2.4 Check resistance to longitudinal shear in SLS

$$v_{L,Ed} \le v_{L,Rd} \tag{2.80}$$

where,

$v_{L,Ed}$: Design longitudinal shear force per unit length at the interface between steel and concrete.

$v_{L,Rd}$: Resistance to longitudinal shear.

2.5 Verification of longitudinal shear in SLS

By Result Table

Verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Longitudinal shear in SLS...

Model View | Longitudinal Shear(SLS)

Elem	Position	Lcom	Type	V_c,Ed (kN)	v_L,Ed (kN/mm)	P_Rd_ser (kN)	v_L,Rd (kN/mm)
2	I[2]	sLCB3	Characteristic	-21604.7029	17.2018	75.3982	0.5027
2	J[3]	-	-	-	-	-	-

V_c,Ed: Vertical shear force acting on the composite section.
v_L,Ed: Longitudinal shear force per unit length in the shear connector.
P_Rd_ser: Shear resistance of a single shear connector for SLS.
v_L,Rd: Longitudinal shear resistance per unit length for the shear connector.

By Excel Report

9 Longitudinal Shear for SLS(Serviceability limit state)

- Shear resistance of a single connector

Load combination name : sLCB3

$P_{Rd,1}$	=	$0.8 \cdot f_u \cdot \pi \cdot d^2 / 4 / \gamma_V$	=	100.531	kN
$P_{Rd,2}$	=	$0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{(f_{ck} \cdot E_{cm})} / \gamma_V$	=	137.253	kN
P_{Rd}	=	$Min(P_{Rd,1}, P_{Rd,2})$	=	100.531	kN
$P_{Rd,ser}$	=	$k_s \cdot P_{Rd}$	=	75.398	kN

where, f_u	=	400.000	MPa	
α	=	1		for $h_{sc}/d > 4$
Num.	=	2		
d	=	20.000	mm	
h_{sc}	=	100.000	mm	
Space	=	300.000	mm	
k_s	=	0.750		

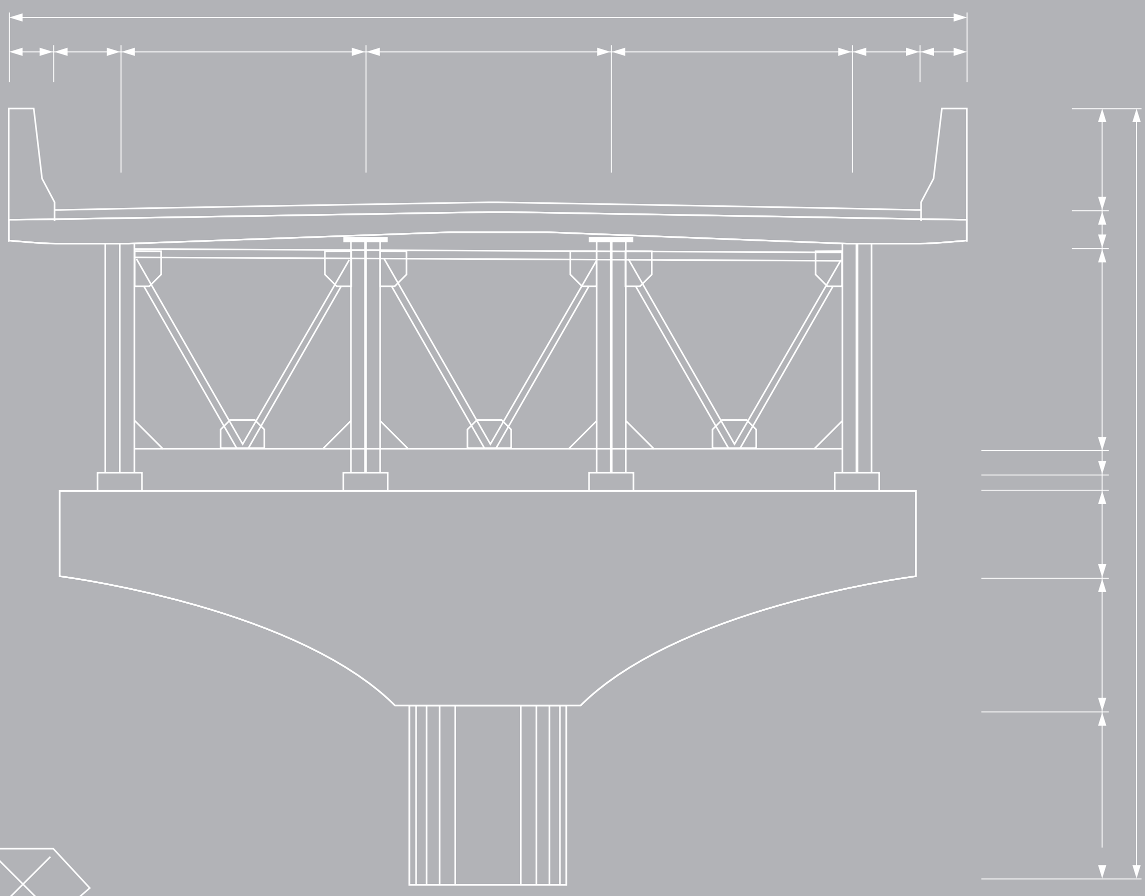
- Verification

$v_{L,Ed}$	=	$V_{Ed} \cdot (A \cdot z / I)$	=	17201.820	kN/m
$v_{L,Rd}$	=	$P_{Rd,ser} \cdot$ Num./Space	=	502.655	kN/m
$v_{L,Ed}$	>	$v_{L,Rd}$	...	NG	

Chapter 3.

Composite Plate
Girder Design

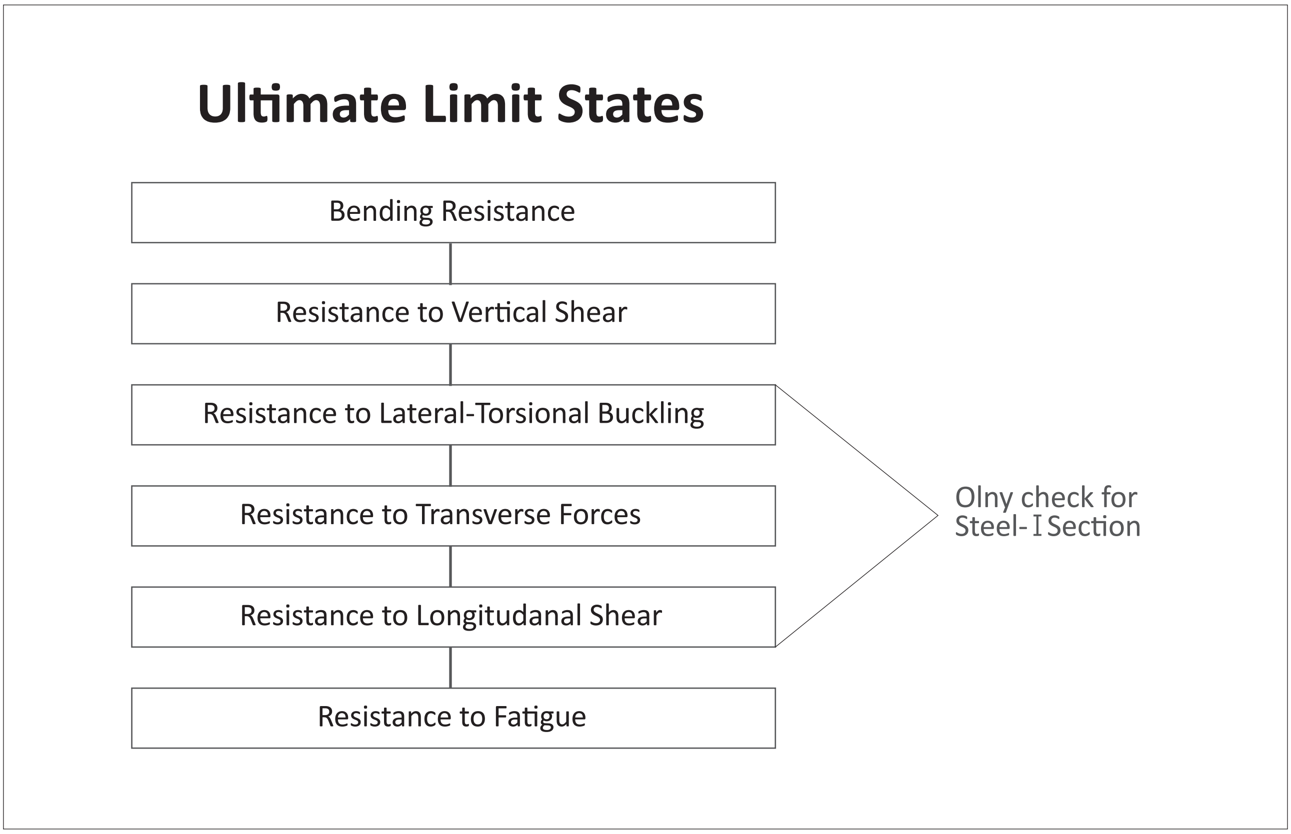
EN 1994-2

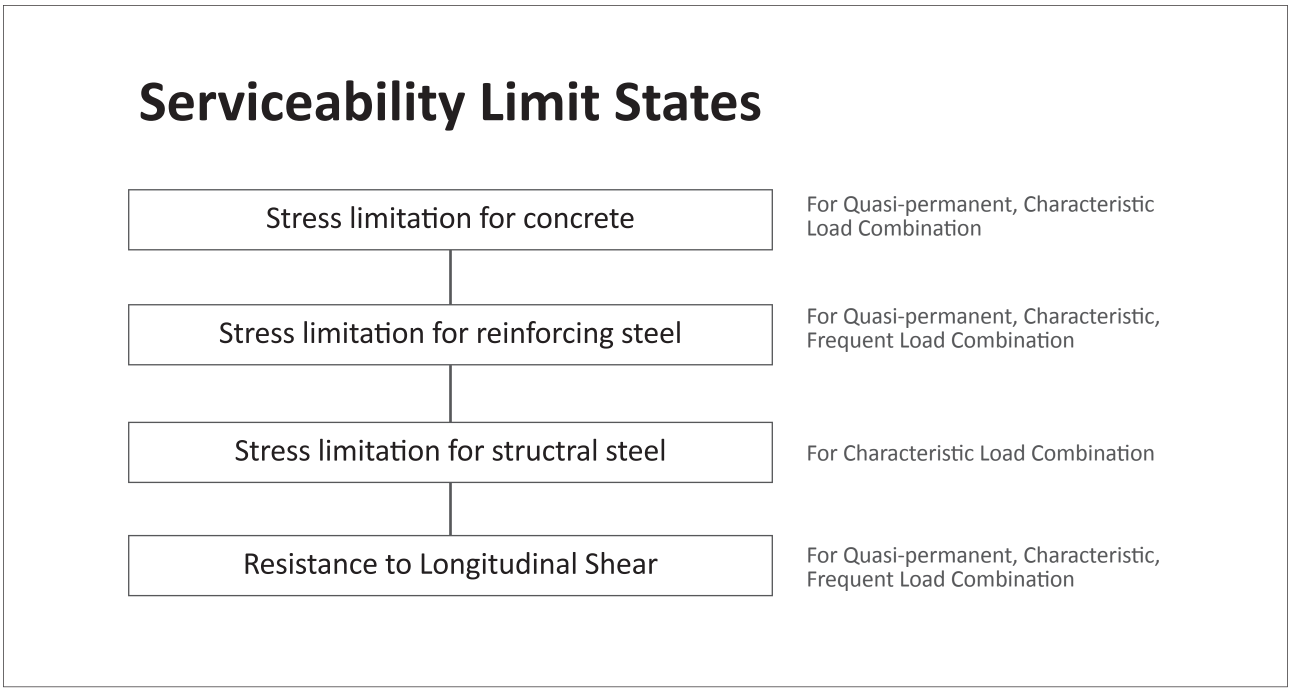


Chapter 3.

Composite Plate Girder Design (EN 1994-2)

Composite Plate needs to be designed to satisfy the following limit states.





Ultimate Limit States

1. Bending resistance

Limit state of Bending Resistance will satisfy the condition, $M_{Ed} \leq M_{Rd}$.
Moment resistance, M_{Rd}, shall be calculated as follows:

1.1 Design values of material

(1) Partial factors for materials

Partial factor for materials considered in ultimate limit states are shown in the table below.

In midas Civil, partial factor for materials can be specified by the user in "Design Parameter" dialog box. The default values are determined as below as per Eurocode 4.

EN1994-2:2005
2.4.1.2

[Table 3.1] Partial factor for materials

Materials	Condition	Partial Factor
Concrete	Persistent & Transient	γ_c = 1.5
	Accidental	γ_c = 1.2
Reinforcing steel	Persistent & Transient	γ_s = 1.15
	Accidental	γ_s = 1.0
Structural steel	Cross-sections	γ_{M0} = 1.0
	Members to instability assessed	γ_{M1} = 1.0
Shear connection	members to instability	γ_V = 1.25
Fatigue verification of headed studs	Strength	γ_{Mf} = 1.0
	Strength of studs in shear	$\gamma_{Mf,s}$ = 1.0

(2) Design compressive strength of concrete.

$$f_{cd} = f_{ck} / \gamma_c \tag{3.1}$$

EN1994-2:2005
(2.1)

where,
f_{ck} : The characteristic compressive cylinder strength of concrete at 28 days.
γ_c : The partial safety factor for concrete.

(3) Design yield strength of steel reinforcement.

$$f_{sd} = f_{sk} / \gamma_s \tag{3.2}$$

where,
f_{sk} : The characteristic value of the yield strength of reinforcing steel.
γ_s : The partial factor for reinforcing steel.

(4) Design yield strength of structural steel.

$$f_{yd} = f_y / \gamma_{M0} \tag{3.3}$$

where,

f_y : *The nominal value of the yield strength of structural steel.*

γ_{M0} : *The partial factor for structural steel applied to resistance of cross-sections.*

The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel shall be obtained by using the simplification given in Fig. 3.1.

EN1993-1-1:2005
Table 3.1

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \le 40$ mm		40 mm $< t \le 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

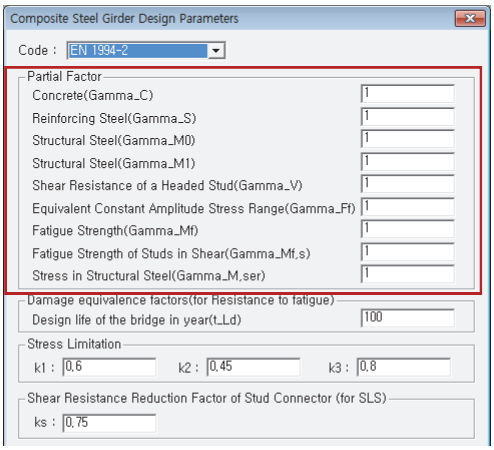
[Fig. 3.1] Nominal values of yield strength f_y and ultimate tensile strength f_u

Partial safety factor

Parameters related to the material such as partial factors, damage equivalence factors, and shear resistance reduction factor can be defined in "Composite Steel Girder Design Parameters" dialog box.

The default values of partial factors are defined as "1.0".

Design > Composite Steel Girder Design>Design Parameters...



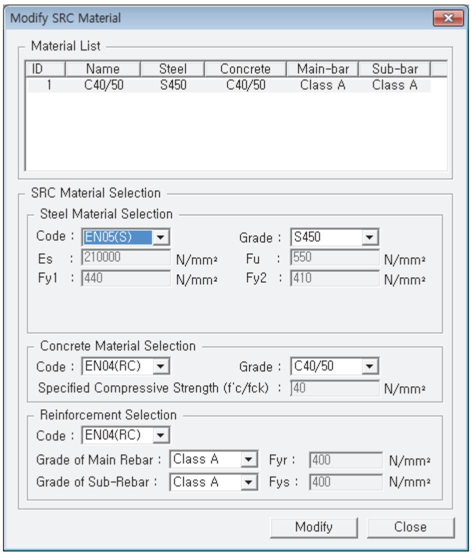
[Fig. 3.2] Design Parameters Dialog

Design strength of materials

Design strength of concrete, reinforcement, and steel can be defined in “Modify SRC Material” dialog box.

In Steel Design Selection field, when Code is entered as “EN05”, F_{y1} is tensile strength of the steel for which the thickness is less or equal to 40mm and F_{y2} is tensile strength of the steel for which the thickness is larger than 40mm.

Design > Composite Steel Girder Design>Design Material…



[Fig. 3.3] Composite steel girder design material

1.2 Classification of cross-section

The classification system defined in EN1993-1-1:2005, 5.5.2 applies to cross-sections of composite beams.

EN1993-1-1:2005
5.5.2

[Table 3.2] Classes of cross-sections

Class	Defined as
1	which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance
2	which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling
3	in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance
4	in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section

(1) The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression.

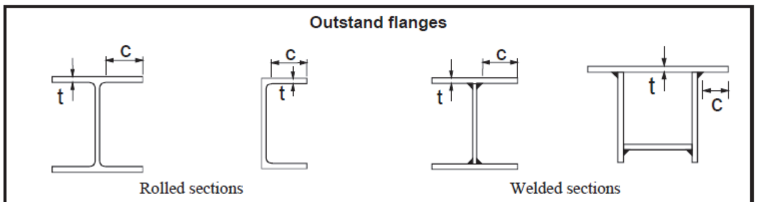
• Classification of Class in flange

Class of flange can be classified depending on the Positive and negative moment.

[Table 3.3] Class of compression flange

Moment	Position	Class of compression flange
Positive	Top Flange	A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class1.
Negative	Bottom Flange	-Composite-I: Check for outstand flanges in Fig. 3.4. -Composite-Box : Check for outstand flanges and internal compression part in Fig. 3.5.

EN1993-1-1:2005
Table 5.2

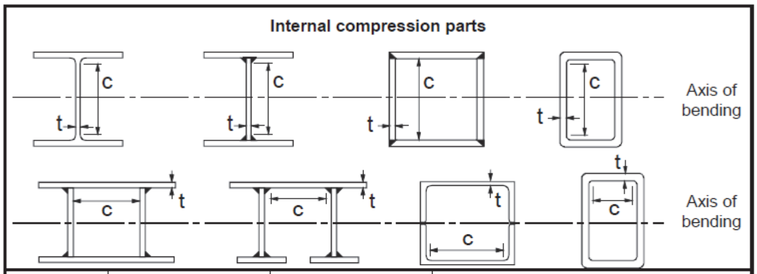


Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

[Fig. 3.4] Maximum width-to-thickness ratios for compression parts - Outstand

• Classification of Class in web: check for internal compression part in Fig. 3.5.



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)	f_y, +, -, c, f_y	f_y, +, -, c, f_y	f_y, +, -, αc, c, f_y
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)	f_y, +, -, c, c/2, f_y	f_y, +, c	f_y, +, -, c, ψf_y
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y/E$

[Fig. 3.5] Maximum width-to-thickness ratios for compression parts - Internal

EN1993-1-1:2005
Table 5.2

(2) Classification of a cross-section

A cross-section is classified according to the highest (least favorable) class of its compression parts as following table.

EN1993-1-1:2005
5.5.2(6)

[Table 3.4] Class of section according to class of compression parts

Class of Section		Class of Flange			
		1	2	3	4
Class of Web	1	1	2	3	4
	2	1	2	3	4
	3	2*	2*	3	4
	4	4	4	4	4

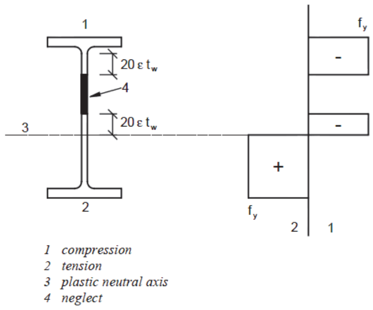
*: Cross-sections with webs in Class3 and flanges in Class1 or 2 may be treated as an effective cross-sections in Class2 with an effective web in accordance with EN1993-1-1:2005, 6.2.2.4. This clause is applied to I-shape section only.

EN1993-1-1:2005
5.5.2(11)

• Effective Class 2 cross-section

The proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange, with another part of $20\varepsilon t_w$ adjacent to the plastic neutral axis of the effective cross-section in accordance with following figure.

EN1993-1-1:2005
6.2.2.4(1)



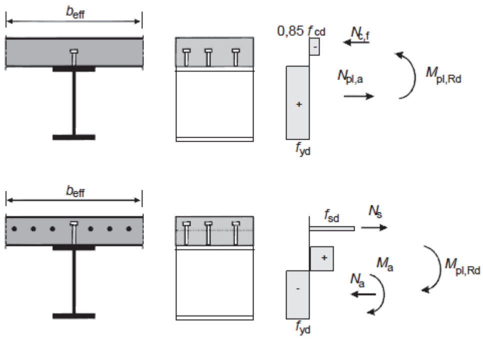
[Fig. 3.6] Effective class 2 web

EN1993-1-1:2005
Figure 6.3

1.3 Calculate plastic bending resistance, $M_{pl,Rd}$.

• For positive moment: Compressive rebar in the deck will be ignored.

• For negative moment: Concrete area of deck will be neglected and only the tensile rebar in the deck will be considered.

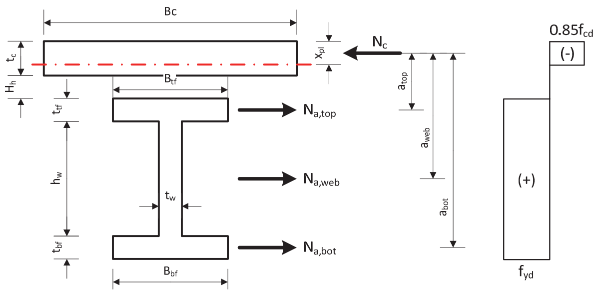


[Fig. 3.7] Plastic stress distributions for a composite beam

EN1994-2:2005
Figure 6.2

For I-shape girder under sagging moment, $M_{pl,Rd}$ can be calculated depending on the position of plastic neutral axis.

(1) Located in the slab depth for positive moment

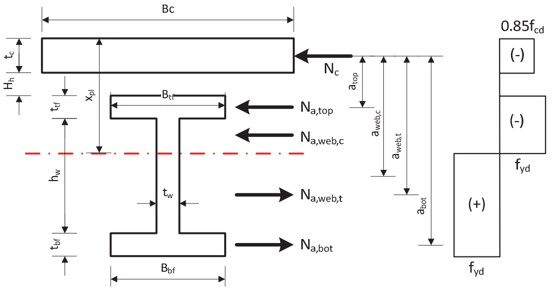


[Fig. 3.8] PNA in the slab depth for positive moment

[Table 3.5] $M_{pl,Rd}$ in the slab depth for positive moment

Part	Force	Distance
Slab	$N_c = 0.85 f_{cd} B_c x_{pl}$	-
Top Flange	$N_{a,top} = f_{yd} b_{tf} t_{tf}$	$a_{top} = t_c + H_h + 0.5t_{tf} - 0.5x_{pl}$
Web	$N_{a,web} = f_{yd} t_w h_w$	$a_{web} = t_c + H_h + t_{tf} + 0.5t_w - 0.5x_{pl}$
Bottom Flange	$N_{a,bot} = f_{yd} b_{bf} t_{bf}$	$a_{bot} = t_c + H_h + t_f + t_w + 0.5t_{bf} - 0.5x_{pl}$
$M_{pl,Rd}$	$M_{pl,Rd} = N_{a,top} a_{top} + N_{a,web} a_{web} + N_{a,bot} a_{bot}$	

(2) Located in the web of steel girder for positive moment



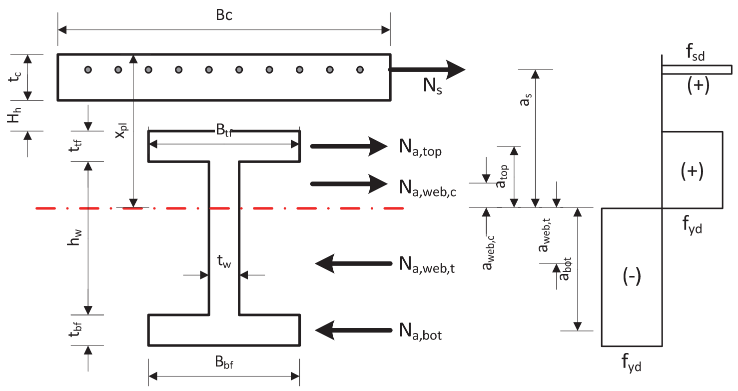
[Fig. 3.9] PNA in the web of steel girder for positive moment

[Table 3.6] $M_{pl,Rd}$ in the web of steel girder for positive moment

Part	Force	Distance
Slab	$N_c = 0.85 f_{cd} B_c t_c$	-
Top Flange	$N_{a,top} = f_{yd} b_{tf} t_{tf}$	$a_{top} = 0.5t_c + H_h + 0.5t_{tf}$
Web (Comp)	$N_{a,web,c} = f_{yd} t_w (x_{pl} - t_c - H_h - t_{tf})$	$a_{web,c} = 0.5(x_{pl} + H_h + t_{tf})$
Web (Tens)	$N_{a,web,t} = f_{yd} t_w (t_c + H_h + t_{tf} + h_w - x_{pl})$	$a_{web,t} = 0.5(x_{pl} + H_h + t_{tf} + h_w)$
Bottom Flange	$N_{a,bot} = f_{yd} b_{bf} t_{bf}$	$a_{bot} = 0.5t_c + H_h + t_{tf} + t_w + 0.5t_{bf}$
$M_{pl,Rd}$	$M_{pl,Rd} = -N_{a,top} a_{top} - N_{a,web,c} a_{web,c} + N_{a,web,t} a_{web,t} + N_{a,bot} a_{bot}$	

For I-shape girder under hogging moment, when plastic neutral axis is located in the web, $M_{pl,Rd}$ can be calculated as follows. The moment is calculated based on the position of plastic neutral axis.

(3) Located in the web of steel girder for negative moment



[Fig. 3.10] PNA in the web of steel girder for negative moment

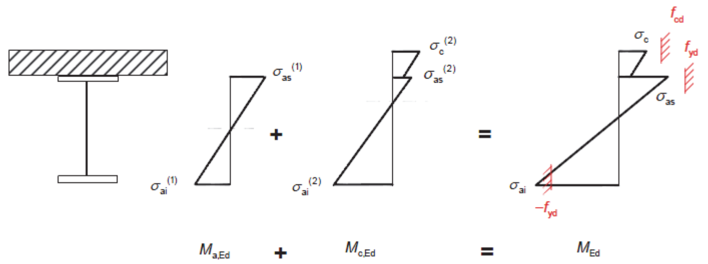
[Table 3.7] $M_{pl,Rd}$ in the web of steel girder for negative moment

Part	Force	Distance
Slab Rebar	$N_{si} = f_{sd} A_{si}$	$a_{si} = x_{pl} - d_{si}$
Top Flange	$N_{a,top} = f_{yd} B_{tf} t_{tf}$	$a_{top} = x_{pl} - t_c - H_h - 0.5t_{tf}$
Web (Tens)	$N_{a,web,t} = f_{yd}(x_{pl} - t_c - H_h - t_{tf}) t_w$	$a_{web,t} = 0.5(x_{pl} + t_c + H_h + t_{tf})$
Web (Comp)	$N_{a,web,c} = f_{yd}(t_c + H_h + t_{tf} + h_w - x_{pl}) t_w$	$a_{web,c} = 0.5(t_c + H_h + t_{tf} + h_w - x_{pl})$
Bottom Flange	$N_{a,bot} = f_{yd} B_{bf} t_{bf}$	$a_{bot} = t_c + H_h + t_{tf} + h_w + 0.5t_{bf} - x_{pl}$
$M_{pl,Rd}$	$M_{pl,Rd} = \sum N_{si} a_{si} + N_{a,top} a_{top} + N_{a,web,t} a_{web,t} + N_{a,web,c} a_{web,c} + N_{a,bot} a_{bot}$	

1.4 Calculate elastic bending resistance, $M_{el,Rd}$

$$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \quad (3.4)$$

EN1994-2:2005 (6.4)



[Fig. 3.11] Calculation of $M_{el,Rd}$

where,

$M_{a,Ed}$: *Design bending moment applied to structural steel section before composite behavior. Bending moment obtained during the construction stage analysis is used in midas Civil.*

$M_{c,Ed}$: *The part of design bending moment acting on the composite section. Bending moment obtained from the final construction stage is used in midas Civil.*

k : *The lowest factor such that a stress limit in EN1994-2:2005, 6.2.1.5(2) is reached. In midas Civil, the value of "k" is calculated as below.*

[Table 3.8] Calculation of k

Type	For Positive Moment	For Negative Moment
Steel Girder	$k_a = \dfrac{f_{yd} - M_{a,Ed}(z_a / I_{y,a})}{M_{c,Ed}(z_c / I_{y,c})}$	$k_a = \dfrac{f_{yd} - M_{a,Ed}(z_a / I_{y,a})}{M_{c,Ed}(z_c / I_{y,c})}$
Slab	$k_c = \dfrac{f_{cd}}{M_{c,Ed}(z_{c,slab} / I_{y,c,slab})}$	-
Reinforcement	-	$k_s = \dfrac{f_{sd}}{M_{c,Ed}(z_{c,bar} / I_{y,c,bar})}$
k	min[k_a, k_c]	min[k_a, k_s]

1.5 Calculate effective cross-section for Class 4 section

(1) Calculate effective cross-section

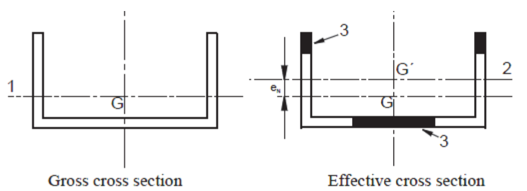
For cross-sections in Class4, the effective structural steel section should be determined in accordance with EN1993-1-5, 4.3.

EN1993-1-5:2006 4.3

In midas Civil, the effect of share lag is not considered in the calculation of effective area. Only the plate buckling effect is considered.

• The effective area A_{eff} should be determined assuming that the cross section is subject only to stresses due to uniform axial compression.

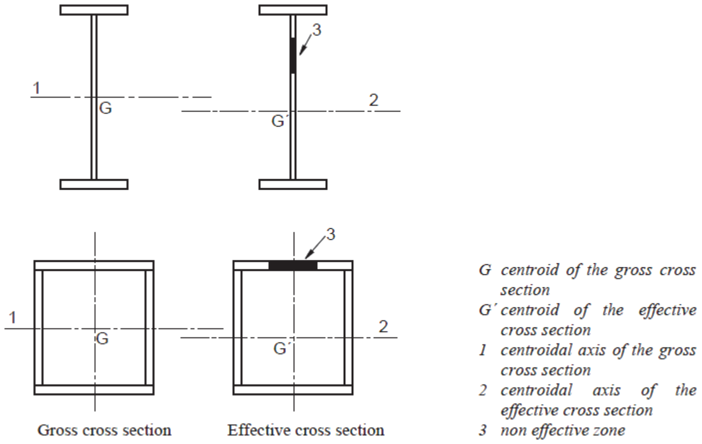
EN1993-1-5:2006 Figure 4.1



G centroid of the gross cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

[Fig. 3.12] Class 4 cross-sections - axial force

• The effective section modulus W_{eff} should be determined assuming that the cross section is subject only to bending stresses.



[Fig. 3.13] Class 4 cross-sections - bending moment

EN1993-1-5:2006 Figure 4.2

The calculation of effective area depending on the longitudinal stiffener will be explained in the clause 1.6 and 1.7 in this manual.

(2) Consideration of additional moment due to the eccentricity of gravity center between the gross area and the effective area

In case of the section with Class 4 classification under the compressive force, the additional moment due to the different gravity center between gross area and effective area is taken into account in the design moment.

EN1993-1-1:2005 6.2.2.5(4)

$$\Delta M_{Ed} = N_{Ed} e_N = N_{Ed}\left(C_{z,c} - C_{z,c,eff}\right) \quad (3.5)$$

where,
e_N : Eccentricity between the gross area and effective area
$C_{z,c}$: Gravity center of the gross area
$C_{z,c,eff}$: Gravity center of the effective area

1.6 Plate elements without longitudinal stiffeners

The effective areas of flat compression elements should be obtained using Table 2.8 for internal elements and Table 2.9 for outstand elements. The effective area of the compression zone of plate should be obtained from :

EN1993-1-5:2006 4.4

$$A_{c,eff} = \rho A_c \quad (3.6)$$

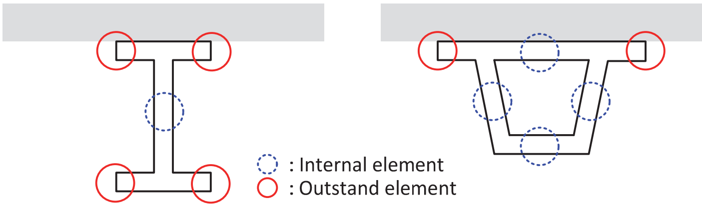
where,
$A_{c,eff}$: Effective cross sectional area.
A_c : The gross cross sectional area.
ρ : The reduction factor for plate buckling.

(1) Effective width b_{eff}

Refer to the following table and figure to see the definition of internal element and outstand element in midas Civil.

[Table 3.9] Definition of internal and outstand element

Type	Shape	Defined as
Internal element	I	Web
	Box	Web / Flanges between web
Outstand element	I	Flange
	Box	Outstand flange which is the outside of webs

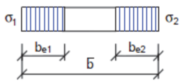
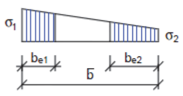
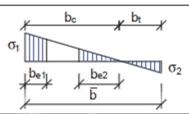


[Fig. 3.14] Internal and outstand element

• For internal compression elements

[Table 3.10] Internal compression elements

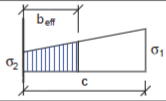
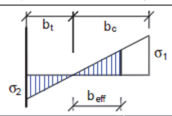
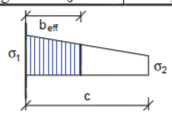
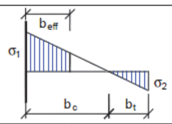
EN1993-1-5:2006 Table 4.1

<table>
<tr><th colspan="4">Stress distribution (compression positive)</th><th colspan="3">Effective^p width b_{eff}</th></tr>
<tr><td colspan="4"></td><td colspan="3">$\underline{\psi = 1}$:
$b_{eff} = \rho\ \bar{b}$
$b_{e1} = 0,5\ b_{eff}$ $b_{e2} = 0,5\ b_{eff}$</td></tr>
<tr><td colspan="4"></td><td colspan="3">$\underline{1 > \psi \geq 0}$:
$b_{eff} = \rho\ \bar{b}$
$b_{e1} = \frac{2}{5-\psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$</td></tr>
<tr><td colspan="4"></td><td colspan="3">$\underline{\psi < 0}$:
$b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$
$b_{e1} = 0,4\ b_{eff}$ $b_{e2} = 0,6\ b_{eff}$</td></tr>
<tr><td>$\psi = \sigma_2/\sigma_1$</td><td>1</td><td>$1 > \psi > 0$</td><td>0</td><td>$0 > \psi > -1$</td><td>-1</td><td>$-1 > \psi > -3$</td></tr>
<tr><td>Buckling factor k_σ</td><td>4,0</td><td>$8,2 / (1,05 + \psi)$</td><td>7,81</td><td>$7,81 - 6,29\psi + 9,78\psi^2$</td><td>23,9</td><td>$5,98\ (1 - \psi)^2$</td></tr>
</table>

• For outstand compression elements

[Table 3.11] Outstand compression elements

EN1993-1-5:2006 Table 4.2

<table>
<tr><th colspan="3">Stress distribution (compression positive)</th><th colspan="3">Effective^p width b_{eff}</th></tr>
<tr><td colspan="3"></td><td colspan="3">$\underline{1 > \psi \geq 0}$:
$b_{eff} = \rho\ c$</td></tr>
<tr><td colspan="3"></td><td colspan="3">$\underline{\psi < 0}$:
$b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$</td></tr>
<tr><td>$\psi = \sigma_2/\sigma_1$</td><td>1</td><td>0</td><td>-1</td><td colspan="2">$1 \geq \psi \geq -3$</td></tr>
<tr><td>Buckling factor k_σ</td><td>0,43</td><td>0,57</td><td>0,85</td><td colspan="2">$0,57 - 0,21\psi + 0,07\psi^2$</td></tr>
<tr><td colspan="3"></td><td colspan="3">$\underline{1 > \psi \geq 0}$:
$b_{eff} = \rho\ c$</td></tr>
<tr><td colspan="3"></td><td colspan="3">$\underline{\psi < 0}$:
$b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$</td></tr>
<tr><td>$\psi = \sigma_2/\sigma_1$</td><td>1</td><td>$1 > \psi > 0$</td><td>0</td><td>$0 > \psi > -1$</td><td>-1</td></tr>
<tr><td>Buckling factor k_σ</td><td>0,43</td><td>$0,578 / (\psi + 0,34)$</td><td>1,70</td><td>$1,7 - 5\psi + 17,1\psi^2$</td><td>23,8</td></tr>
</table>

(2) Reduction factor ρ

EN1993-1-5:2006 4.4(2)

[Table 3.12] Calculation of reduction factor ρ

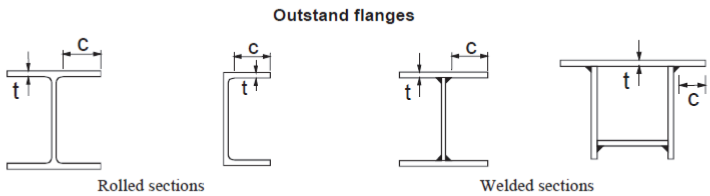
Type	Condition	ρ
Internal element	$\overline{\lambda}_p \le 0.673$	1.0
	$\overline{\lambda}_p > 0.673$ where, $(3+\psi) \ge 0$	$\dfrac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^{\,2}} \le 1.0$
Outstand element	$\overline{\lambda}_p \le 0.748$	1.0
	$\overline{\lambda}_p > 0.748$	$\dfrac{\overline{\lambda}_p - 0.188}{\overline{\lambda}_p^{\,2}} \le 1.0$

where,

$$\overline{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\overline{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} \tag{3.7}$$

$\overline{b}$: *The appropriate width to be taken as follow.*
b_w : *For webs*
b : *For internal flange elements.*
c : *For outstand flanges.*

EN1993-1-1:2005 Table 5.2



[Fig. 3.15] Dimension of outstand flanges

Ψ : *The stress ratio.*
k_σ : *The buckling factor corresponding to the stress ratio ψ and boundary conditions.*
t : *The thickness.*
σ_{cr} : *The elastic critical plate buckling stress.*

$$\varepsilon = \sqrt{\frac{235}{f_y[\mathrm{N/mm^2}]}} \tag{3.8}$$

1.7 Stiffened plate elements with longitudinal stiffeners

The effective section area of each subpanel should be determined by a reduction factor in accordance with 1.6 to account for local buckling. The stiffened plate with effective section area for the stiffeners should be checked for global plate buckling and a reduction factor ρ should be determined for overall plate buckling.

EN1993-1-5:2006 4.5

The effective area of the compression zone of the stiffened plate should be taken as:

$$A_{c,eff} = \rho_c A_{c,eff,loc} + \sum b_{edge,eff} t \tag{3.9}$$

EN1993-1-5:2006 (4.5), (4.6)

$$A_{c,eff,loc} = A_{sl,eff} + \sum_c \rho_{loc} b_{c,loc} t \tag{3.10}$$

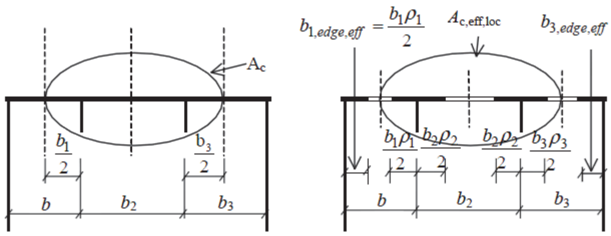
where,

$A_{c,eff,loc}$: *The effective section areas of all the stiffeners and subpanels that are fully or partially in the compression zone except the effective parts supported by an adjacent plate element with the width $b_{edge,eff}$.*

$\sum_c$ *applies to the part of effective section according to 1.6 of all longitudinal stiffeners with gross area A_{sl} located in the compression zone.*

$b_{c,loc}$: *The width of the compressed part of each subpanel.*

ρ_{loc} : *The reduction factor from 1.5 for each subpanel.*



[Fig. 3.16] Stiffened plate under uniform compression

EN1993-1-5:2006 Figure 4.4

(1) Effective width and reduction factor for individual subpanels between stiffeners.
Calculate the effective width of subpanels between stiffeners as per the clause 1.6.

The value of $\bar{b}$ is taken as the smaller value between the follows:

-Clear spacing between flange and stiffener

-Clear spacing between stiffeners

(2) Elastic critical plate buckling stress $\sigma_{cr,p}$ for stiffened web.

• with single stiffener in the compression zone

$\sigma_{cr,p}$ can be calculated as follows ignoring stiffeners in the tension zone :

$$\sigma_{cr,p} = \sigma_{cr,sl} \tag{3.11}$$

EN1993-1-5:2006 A.2.2(1)

[Table 3.13] Calculation of $\sigma_{cr,sl}$

Condition	$\sigma_{cr,sl}$
$a \geq a_c$	$\frac{1.05E}{A_{sl,1}}\frac{\sqrt{I_{sl,1}t^3b}}{b_1b_2}$
$a < a_c$	$\frac{\pi^2 EI_{sl,1}}{A_{sl,1}a^2} + \frac{Et^3ba^2}{4\pi^2(1-\nu^2)A_{sl,1}b_1^2b_2^2}$

EN1993-1-5:2006 (A.4)

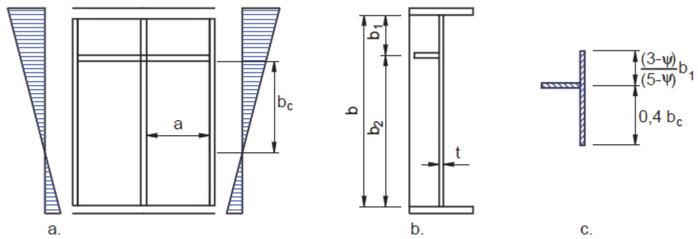
where,

$$a_c = 4.33\sqrt[4]{\frac{I_{sl,1}b_1^2b_2^2}{t_3b}} \tag{3.12}$$

$A_{sl,1}$: *The gross area of the column.*

$I_{sl,1}$: *The second moment of area of the gross cross-section of the column.*

b_1, b_2 : *The distances from the longitudinal edges of the web to the stiffener.*



[Fig. 3.17] Notations for a web plate with single stiffener in the compression zone

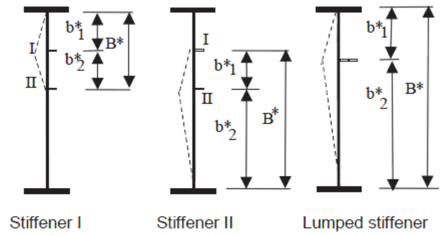
EN1993-1-5:2006 Figure A.2

• with two stiffeners in the compression zone

$\sigma_{cr,p}$ should be taken as the lowest of those computed for the 3 cases using equation (3.13) with $b_1=b_1^*$, $b_2=b_2^*$, $b=B^*$. The stiffeners in tension zone should be ignored.

$$\sigma_{cr,p} = \min\left[\sigma_{cr,sl,I}, \sigma_{cr,sl,II}, \sigma_{cr,sl,lumped}\right] \tag{3.13}$$

EN1993-1-5:2006 A.2.1(7)



EN1993-1-5:2006 Figure A.3

	Stiffener I	Stiffener II	Lumped stiffener
Cross-sectional area	$A_{s\ell,1}$	$A_{s\ell,2}$	$A_{s\ell,1}+A_{s\ell,2}$
Second moment of area	$I_{s\ell,1}$	$I_{s\ell,2}$	$I_{s\ell,1}+I_{s\ell,2}$

[Fig. 3.18] Notations for plate with two stiffeners in the compression zone

It is assumed that one of stiffeners buckles while the other one acts as a rigid support.

Buckling of both the stiffeners simultaneously is accounted for by considering a single lumped stiffener that is substituted for both individual ones such that :

(a) Its cross-sectional area and its second moment of area I_{st} are respectively the sum of for the individual stiffeners.

(b) It is positioned at the location of the resultant of the respective forces in the individual stiffeners.

• with at least three stiffeners in the compression zone

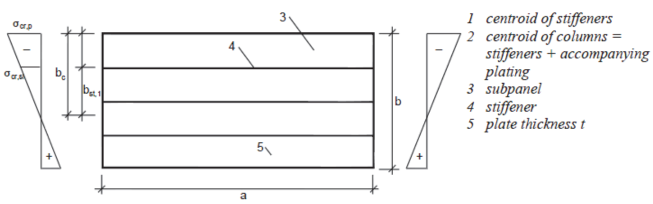
$$\sigma_{cr,p} = k_{\sigma,p}\sigma_E \tag{3.14}$$

EN1993-1-5:2006 A.1(2)

where,

$$\sigma_E = \frac{\pi^2 E t^2}{12(1-\nu^2)b^2} \tag{3.15}$$

$k_{\sigma,p}$ *: The buckling coefficient.*
b is defined in Fig. 3.19.
t : The thickness of the plate.
E : The modulus of elasticity of structural steel.
v : The poisson's ratio



EN1993-1-5:2006 FigureA.1

[Fig. 3.19] Notations for longitudinally stiffened plates (1)

$k_{\sigma,p}$ *may be approximated as in the following table.*

[Table 3.14] Calculation of $k_{\sigma,p}$

Condition	$k_{\sigma,p}$
$\alpha \le \sqrt[4]{\gamma}$	$\dfrac{2\left(\left(1+\alpha^2\right)^2+\gamma-1\right)}{\alpha^2(\psi+1)(1+\delta)}$
$\alpha > \sqrt[4]{\gamma}$	$\dfrac{4\left(1+\sqrt{\lambda}\right)}{(\psi+1)(1+\delta)}$

EN1993-1-5:2006 (A.2)

where,

$$\psi = \frac{\sigma_2}{\sigma_1} \ge 0.5 \tag{3.16}$$

$$\gamma = \frac{\sum I_{sl}}{I_p} \tag{3.17}$$

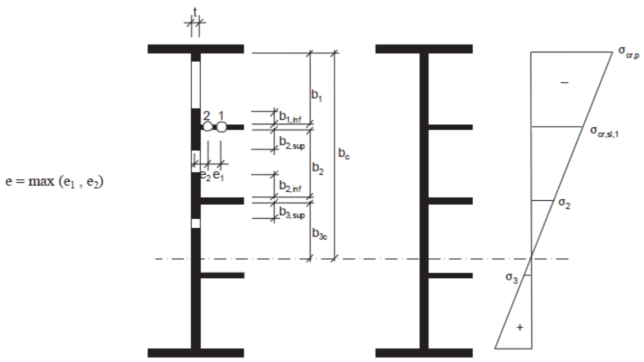
$$\delta = \frac{\Sigma A_{sl}}{A_p} \tag{3.18}$$

$$\alpha = \frac{a}{b} \ge 0.5 \tag{3.19}$$

$\sum I_{sl}$: *The sum of the second moment of area of the whole stiffened plate.*
$\sum A_{sl}$: *The sum of the gross area of the individual longitudinal stiffener.*
I_p : *The second moment of area for bending of the plate.*

$$I_p = \frac{bt^3}{12(1-\upsilon^2)} \tag{3.20}$$

A_p : *The gross area of the plate = bt.*
σ_1 : *The larger edge stress.*
σ_2 : *The smaller edge stress.*
a, b, t : as defined in Fig.3.20.



	width for gross area	width for effective area according to Table 4.1	condition for ψ_i
$b_{1,inf}$	$\dfrac{3-\psi_1}{5-\psi_1} b_1$	$\dfrac{3-\psi_1}{5-\psi_1} b_{1,eff}$	$\psi_1 = \dfrac{\sigma_{cr,sl,1}}{\sigma_{cr,p}} > 0$
$b_{2,sup}$	$\dfrac{2}{5-\psi_2} b_2$	$\dfrac{2}{5-\psi_2} b_{2,eff}$	$\psi_2 = \dfrac{\sigma_2}{\sigma_{cr,sl,1}} > 0$
$b_{2,inf}$	$\dfrac{3-\psi_2}{5-\psi_2} b_2$	$\dfrac{3-\psi_2}{5-\psi_2} b_{2,eff}$	$\psi_2 > 0$
$b_{3,sup}$	$0{,}4\, b_{c3}$	$0{,}4\, b_{c3,eff}$	$\psi_3 = \dfrac{\sigma_3}{\sigma_2} < 0$

[Fig. 3.20] Notations for longitudinally stiffened plates (2)

EN1993-1-5:2006 FigureA.1

(3) Plate type behavior.

• The relative plate slenderness $\overline{\lambda}_p$ of the equivalent plate

$$\overline{\lambda}_p = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,p}}} \tag{3.21}$$

EN1993-1-5:2006 4.5.2(1)

where,

$$\beta_{A,c} = \frac{A_{c,eff,loc}}{A_c} \tag{3.22}$$

A_c : *The gross area of the compression zone of the stiffened plate except the parts of subpanels supported by an adjacent plate.*

$A_{c,eff,loc}$: *The effective area of the same part of the plate with due allowance made for possible plate buckling of subpanels and/or of stiffened panels.*

• The reduction factor ρ

[Table 3.15] Calculation of ρ

Element	Condition	ρ
Internal element	$\overline{\lambda}_p \le 0.673$	1.0
	$\overline{\lambda}_p > 0.673$ where, $(3+\psi) \ge 0$	$\frac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^2} \le 1.0$
Outstand element	$\overline{\lambda}_p \le 0.748$	1.0
	$\overline{\lambda}_p > 0.748$	$\frac{\overline{\lambda}_p - 0.188}{\overline{\lambda}_p^2} \le 1.0$

EN1993-1-5:2006 4.4(2)

(4) Column type behavior.

• The elastic critical column buckling stress $\sigma_{cr,c}$

(a) Unstiffened plate : $\sigma_{cr,c} = \frac{\pi^2 E t^2}{12(1-\nu^2)a^2}$ (3.23)

(b) Stiffened plate : $\sigma_{cr,c} = \sigma_{cr,sl} \frac{b_c}{b_{sl,1}}$ (3.24)

EN1993-1-5:2006 4.5.3(2),(3)

where,

a : Length of a stiffened or unstiffened plate.

$$\sigma_{cr,sl} = \frac{\pi^2 E I_{sl,1}}{A_{sl,1} a^2} \tag{3.25}$$

Isl,1 : The second moment of area of the stiffener, relative to out-of-plane bending of the plate.

Asl,1 : The gross cross-sectional area of the stiffener and the adjacent parts of the plate

- The relative column slenderness $\overline{\lambda}_c$

(a) Unstiffened plate : $\overline{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr,c}}}$ (3.26)

EN1993-1-5:2006 4.5.3(4)

(b) Stiffened plate : $\overline{\lambda}_c = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,c}}}$ (3.27)

where,

$$\beta_{A,c} = \frac{A_{sl,1,eff}}{A_{sl,1}} \quad (3.28)$$

$A_{sl,1,eff}$ *: The effective cross-sectional area of the stiffener with due allowance for plate buckling.*

- The reduction factor χ_c

$$\chi_c = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}_c^2}} \le 1.0 \quad (3.29)$$

EN1993-1-1:2005 6.3.1.2

(a) Unstiffened plate : $\Phi = 0.5\left[1 + \alpha\left(\overline{\lambda}_c - 0.2\right) + \overline{\lambda}_c^2\right]$ (3.30)

where, $\alpha = 0.21$

(b) Stiffened plate : $\Phi = 0.5\left[1 + \alpha_e\left(\overline{\lambda}_c - 0.2\right) + \overline{\lambda}_c^2\right]$ (3.31)

where,

$$\alpha_e = \alpha + \frac{0.09}{i/e} \quad (3.32)$$

EN1993-1-5:2006 4.5.3(5)

$$i = \sqrt{\frac{I_{sl,1}}{A_{sl,1}}} \quad (3.33)$$

e = max(e1, e2) is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the column.
α = 0.34 (for closed section stiffener), 0.49 (for open section stiffener)

(5) Final reduction factor ρ_c from interaction between plate and column buckling.

EN1993-1-5:2006 4.5.4(1)

$$\rho_c = (\rho - \chi_c)\xi(2 - \xi) + \chi_c \quad (3.34)$$

where,

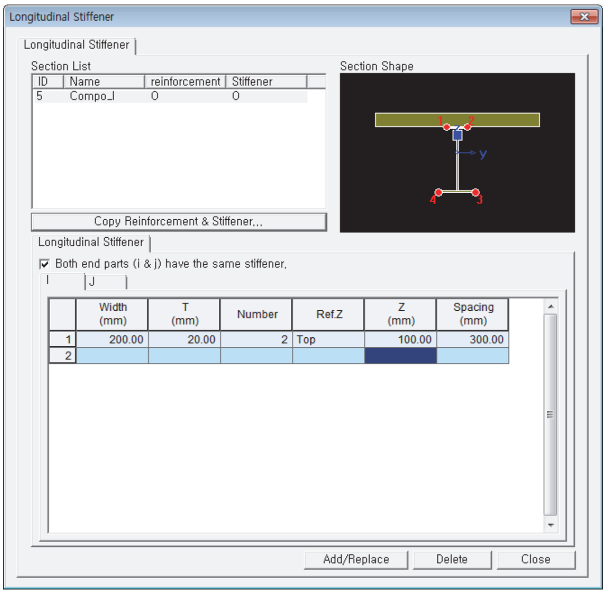
$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1, \quad 0 \le \xi \le 1.0 \quad (3.35)$$

$\sigma_{cr,p}$ *: The elastic critical plate buckling stress.*
$\sigma_{cr,c}$ *: The elastic critical column buckling stress.*
χ_c *: The reduction factor due to column buckling.*

Longitudinal stiffener

Longitudinal stiffeners of plate girder need to be entered by section properties. Flat type stiffener can be defined.

Design>Composite Steel Girder Design>Longitudinal Stiffener(Plate Girder Only)...



[Fig. 3.21] Longitudinal stiffener Dialog

1.8 Calculate bending resistance, M_{Rd}

Bending resistance, M_{Rd}, can be calculated as follows based on its class.

Class 1 or 2 cross-sections can be checked by using the plastic or elastic bending resistance.

Class 3 cross-sections are checked with the elastic bending resistance, or possibly reclassified as effective Class 2 cross-section and then checked with the plastic bending resistance.

Class 4 cross-sections are also checked with the elastic bending resistance but by using the effective cross-section, reduced to take account of buckling.

(1) Class 1 and 2 + Positive Moment.

• The strength of the reinforcing steel bars in compression is neglected.

• General case : $M_{Rd} = M_{pl,Rd}$ (3.36)

• For the structural steel grade S420 or S460, M_{Rd} is calculated as shown in the table below.

[Table 3.16] Calculation M_{Rd}

Condition		M_{Rd}
$x_{pl} \le 0.15h$		$M_{pl,Rd}$
$0.15h < x_{pl} \le 0.4h$		$\beta M_{pl,Rd}$
$x_{pl} > 0.4h$	$N_c \le N_{c,el}$	$M_{a,Ed} + (M_{el,Rd} - M_{a,Ed})\frac{N_c}{N_{c,el}}$
	$N_{c,el} < N_c \le N_{c,f}$	$M_{el,Rd} + (M_{pl,Rd} - M_{el,Ed})\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}}$

EN1994-2:2005
6.2.1.4(6)

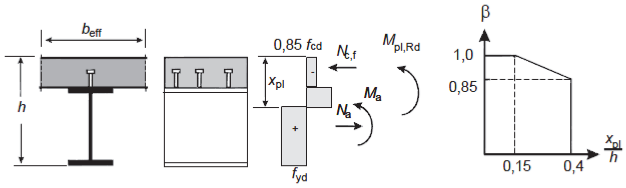
where,

$M_{pl,Rd}$: *Design value of the plastic resistance moment of the composite section with full shear connection.*

$M_{el,Rd}$: *Design value of the elastic resistance moment of the composite section.*

β : *The reduction factor.*

N_c : *Design value of the compressive normal force in the concrete flange.*
$N_{c,el}$: *Compressive normal force in the concrete flange corresponding to* $M_{el,Rd}$.
$N_{c,f}$: *Design value of the compressive normal force in the concrete flange with full shear connection.*



[Fig. 3.22] Reduction factor β for $M_{pl,Rd}$

(2) Class 1 and 2 + Negative Moment.

• The strength of the concrete in tension is neglected.

• Bending resistance

$$M_{Rd} = M_{pl,Rd} \tag{3.37}$$

(3) Class 3

• Bending resistance

$$M_{Rd} = M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \tag{3.38}$$

(4) Class 4

• Section properties should be calculated by considering the effective area. If the section is under the compression, the additional moment must be taken in to account due to the eccentricity between the gravity center of gross section and effective section.

EN1994-2:2005 (6.4)

Refer to the clause 1.5 to see how to calculate the effective area and additional moment.

• Bending resistance

$$M_{Rd} = M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \tag{3.39}$$

EN1994-2:2005 (6.4)

1.9 Check bending resistance

$$M_{Ed} \le M_{Rd} \tag{3.40}$$

where,
M_{Ed} : *Design bending moment.*
M_{Rd} : *Design moment resistance.*

• Load combination

In midas Civil, bending resistance will be verified for the load combinations that the Active column is specified as Strength/Stress in Results>Load combinations>Steel Design tab.

1.10 Verification of bending resistance

By Result Table

Bending resistance can be verified in the table format as shown below.

Design>Composite Steel Girder Design>Design Result Tables>Bending Resistance...

Model View | **Bending Resistance**

	Elem	Position	Positive/Negative	Lcom	Type	Top Class	Bot Class	Web Class	Sect. Class	Ma,Ed (kN·m)	Mc,Ed (kN·m)	Mpl,Rd (kN·m)	Mel,Rd (kN·m)	M_Rd (kN·m)
▶	2	I[2]	Negative	sLCB1	-	1	2	1	2	-36.4051	-1240.8507	14706.4069	7821.1822	14706.4069
	2	I[2]	Positive	-	-	-	-	-	-	-	-	-	-	-
	2	J[3]	Negative	-	-	-	-	-	-	-	-	-	-	-
	2	J[3]	Positive	-	-	-	-	-	-	-	-	-	-	-

Positive/Negative: Positive/Negative moment
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
Top Class: Class of top flange
Bot Class: Class of bottom flange
Web Class: Class of web
Sect. Class: Class of cross section
Ma,Ed: The design bending moment applied to structural steel section before composite behavior
Mc,Ed: The part of the design bending moment acting on the composite section
Mpl,Rd: Design value of the plastic resistance moment of the composite section
Mel,Rd: Design value of the elastic resistance moment of the composite section
M_Rd: Design value of the resistance moment of a composite section

By Excel Report

Detail results with applied equations and parameters can be checked in the Excel Report.

44	2 Bending Resistance												
45	2.1 Negative Moment												
46	- Design load												
47	Load combination name :	sLCB1											
48	$N_{a,Ed}$	=	0.000	kN									
49	$N_{c,Ed}$	=	-2000.000	kN									
50	$M_{a,Ed}$	=	-36.405	kN · m									
51	$M_{c,Ed}$	=	-1240.851	kN · m									
52													
53	- Stress												
54	Top Flange												
55	Left	y_1	-200.000	mm	z_1	607.710	mm	σ_1	23.487	MPa			
56		y_2	-15.000	mm	z_2	607.710	mm	σ_2	23.487	MPa			
57	Right	y_1	200.000	mm	z_1	607.710	mm	σ_1	23.487	MPa			
58		y_2	15.000	mm	z_2	607.710	mm	σ_2	23.487	MPa			
59													
70	- Classification of sections												
71	Part	Class											
72	Top flange	1											
73	Web	1											
74	Bottom flange	2											
75	Section	2											
76													
77	- Plastic resistance moment, $M_{pl,Rd}$												
78	Plastic NA	=	613.574	mm									
79													
80	N_{slab}	=	0.000	kN									
81	N_{rebar}	=	2895.429	kN									
82	$N_{g,top}$	=	14158.536	kN	(Upper side of PNA)								
83	$N_{g,bot}$	=	17053.964	kN	(Lower side of PNA)								
84													
85	$M_{pl,Rd}$	=	14706.407	kN · m									
86	M_{Rd}	=	$M_{pl,Rd}$	=	14706.407	kN · m							
87													
88	M_{Rd}	=	14706.407	kN · m	>	M_{Ed}	=	-1277.256	kN · m	...OK			

2. Resistance to vertical shear

Limit state of vertical shear resistance will satisfy the condition, $V_{Ed} \leq V_{Rd}$.

Shear resistance, V_{Rd} , will be determined as smaller value between $V_{pl,Rd}$ and $V_{b,Rd}$ when considering shear buckling. When the shear buckling is not considered, Shear resistance, V_{Rd} , will be determined as $V_{pl,Rd}$. The plastic resistance and buckling resistance are calculated as follows:

2.1 Plastic resistance to vertical shear

$$V_{pl,Rd} = V_{pl,a,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} \qquad (3.41)$$

EN1994-2:2005 6.2.2.2 EN1993-1-1:2005 (6.18)

where,

γ_{M0} *: The partial factor for resistance of cross-sections whatever the class is.*

A_v *: The shear area. In midas Civil, only welded I, H and box sections are considered.*

$$A_v = \eta \sum (h_w t_w) \qquad (3.42)$$

EN1993-1-1:2005 6.2.6(3)-d)

h_w *: The depth of the web*

t_w *: The web thickness*

η *: The coefficient that includes the increase of shear resistance at web slenderness*

[Table 3.17] Coefficient η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

2.2 Shear buckling resistance

Plates with $\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$ for an unstiffened web, or $\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$ for a stiffened web, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports.

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \qquad (3.43)$$

EN1994-2:2005 6.2.2.3 EN1993-1-5:2006 (5.1)

(1) Contribution from the web $V_{bw,Rd}$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \qquad (3.44)$$

EN1993-1-5:2006 (5.2)

where,

f_{yw} *: Yield strength of the web.*

h_w *: Clear web depth between flanges.*

t *: Thickness of the plate.*

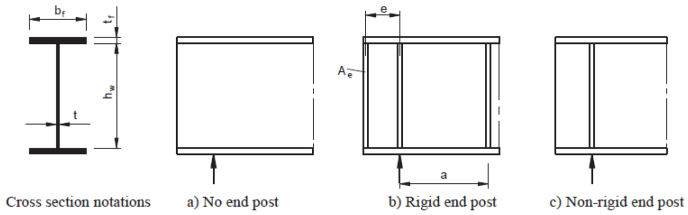
γ_{M1} *: Partial factor for resistance of members to instability assessed by member checks.*

χ_w *: Factor for the contribution of the web to the shear buckling resistance.*

[Table 3.18] Contribution from the web χ_w

Condition	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \le \bar{\lambda}_w < 1.08$	$0.83/\bar{\lambda}_w$	$0.83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1.08$	$1.37/(0.7+\bar{\lambda}_w)$	$0.83/\bar{\lambda}_w$

EN1993-1-5:2006
Table 5.1



[Fig. 3.23] End supports

EN1993-1-5:2006
Figure 5.1

λ_w : Slenderness parameter.

[Table 3.19] Calculation of λ_w

Condition	$\bar{\lambda}_w$
Transverse stiffeners at supports only. (In midas Civil, when longitudinal stiffener exists only)	$\bar{\lambda}_w = \dfrac{h_w}{86.4t\varepsilon}$
Transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both (In midas Civil, except for the condition when longitudinal stiffener exists only)	$\bar{\lambda}_w = \dfrac{h_w}{37.4t\varepsilon\sqrt{k_\tau}}$

EN1993-1-5:2006
5.3(3)

For webs with longitudinal stiffeners,

$$\bar{\lambda}_w \ge \frac{h_{wi}}{37.4t\varepsilon\sqrt{k_{\tau i}}} \tag{3.45}$$

EN1993-1-5:2006
5.3(5)

h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter λ_w of all subpanels within the web panel under consideration. ($k_{\tau st}$ = 0)

$$\varepsilon = \sqrt{\frac{235}{f_y}} \tag{3.46}$$

k_τ : The minimum shear buckling coefficient for the web panel.

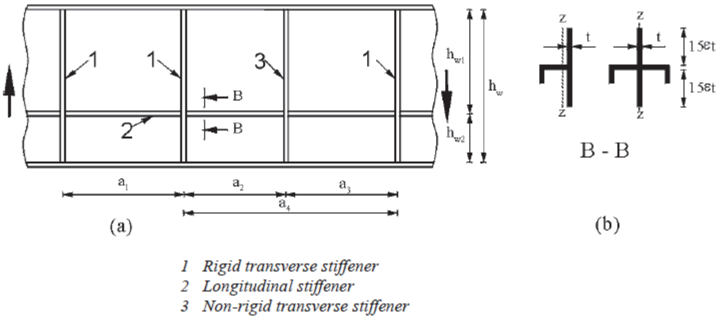
[Table 3.20] Calculation k_τ

Longitudinal stiffeners num.	Condition	k_τ
= 0 or >2	$a/h_w \ge 1.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$a/h_w < 1.0$	$k_\tau = 4.00 + 5.34(h_w/a)^2 + k_{\tau st}$
1 or 2	$\alpha = a/h_w \ge 3.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$\alpha = a/h_w < 3.0$	$k_\tau = 4.1 + \dfrac{6.3 + 0.18\dfrac{I_{sl}}{t^3 h_w}}{\alpha^2} + 2.2\sqrt[3]{\dfrac{I_{sl}}{t^3 h_w}}$

EN1993-1-5:2006
A.3

$$k_{\tau st} = 9\left(\frac{h_w}{a}\right)^2 \sqrt[4]{\left(\frac{I_{sl}}{t^3 h_w}\right)^3} \geq \frac{2.1}{t}\sqrt[3]{\frac{I_{sl}}{h_w}} \quad (3.47)$$

a : The distance between transverse stiffeners.



EN1993-1-5:2006 Figure 5.3

[Fig. 3.24] Web with transverse and longitudinal stiffeners

I_{sl} : *The second moment of area of the longitudinal stiffener about z-axis. The value of* I_{sl} *will be multiplied by 1/3 when calculating* k_τ.

η : *The coefficient that includes the increase of shear resistance at web slenderness*

[Table 3.21] Calculation η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

(2) Contribution from the flange $V_{bf,Rd}$

EN1993-1-5:2006 5.4(1)
EN1993-1-5:2006 (5.8)

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c\gamma_{M1}}\left[1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right] \quad (3.48)$$

where,
b_f *and* t_f *are taken for the flange which provides the least axial resistance.*
b_f *being taken as not larger than* $15\varepsilon_{tf}$ *on each side of the web.*
f_{yf} *: Yield strength of the flange.*

$$c = a\left(0.25 + \frac{1.6 b_f t_f^2 f_{yf}}{t h_w^2 f_{yw}}\right) \quad (3.49)$$

γ_{M1} *: Partial factor for resistance of members to instability assessed by member checks.*
M_{Ed} *: Design bending moment.*
$M_{f,Rd}$ *: The moment of resistance of the cross section consisting of the area of the effective flanges only.*

[Table 3.22] Calculation of $M_{f,Rd}$

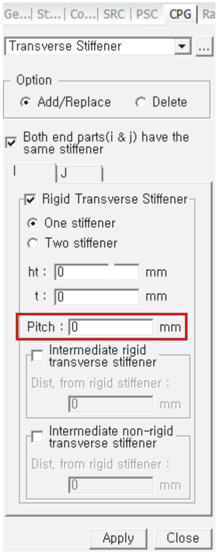
Condition	$M_{f,Rd}$
$N_{Ed} = 0$	$M_{f,Rd}$ is calculated as $M_{pl,Rd}$ but neglecting the web contribution.
N_{Ed} is present	It is calculated by multiplying the reduction factor from the value of $M_{f,Rd}$ when N_{Ed}=0. $1-\frac{N_{Ed}}{\frac{(A_{f1}+A_{f2})f_{yf}}{\gamma_{M0}}}$

EN1993-1-5:2006 (5.9)

Transverse stiffener

Transverse stiffeners can be specified by members.

Design>Composite Steel Girder Design>Transverse Stiffener...

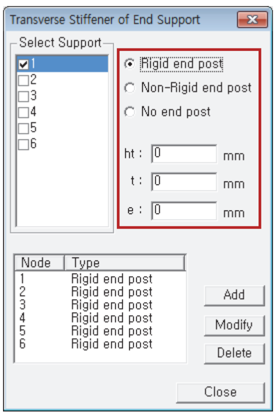


[Fig. 3.25] Transverse stiffener

Transverse stiffener of end support

Transverse stiffener of end support can be entered from the following dialog box. End support type by nodes and related parameter can be defined.

Design>Composite Steel Girder Design>Transverse Stiffener of End Support...



[Fig. 3.26] Transverse Stiffener of End Support

2.3 Resistance to vertical shear

V_{Rd} is calculated depending on the value of h_w/t as shown in the table below.

[Table 3.23] Calculation of V_{Rd}

	Condition	V_{Rd}
Unstiffened	$\frac{h_w}{t} \le \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{b,Rd}$
Stiffened	$\frac{h_w}{t} \le \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{b,Rd}$

where,

$V_{pl,Rd}$: *The plastic resistance to vertical shear.*

$V_{b,Rd}$: *The shear buckling resistance.*

2.4 Interaction bending and vertical shear

EN1994-2:2005 6.2.2.4(1)

(1) Verification condition of interaction between sear force and bending moment

When the following condition is satisfied, combined effects of bending and shear need to be verified.

$$\overline{\eta_3} = \frac{V_{Ed}}{V_{bw,Rd}} > 0.5 \quad (3.50)$$

where,

V_{Ed} : *The design shear force including shear from torque.*

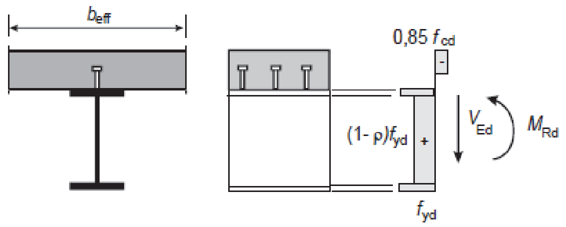
$V_{bw,Rd}$: *The design resistance for shear of contribution from the web.*

(2) For cross-sections in Class1 or 2

Apply the reduced design steel strength $(1-\rho)f_{yd}$ in the shear area.

$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2 \quad (3.51)$$

EN1994-2:2005 6.2.2.4(2) Figure 6.7



[Fig. 3.27] Plastic stress distribution modified by the effect of vertical shear

(3) For cross-sections in Class3 and 4

- $\bar{\eta}_3 \le 0.5$: M_{Rd}, N_{Rd} need not be reduced.
- $\bar{\eta}_3 > 0.5$: The combined effects of bending and shear in the web of an I or box girder should satisfy.

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \le 1.0 \qquad (3.52)$$

EN1993-1-5:2006 7.1(1)

where,

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}} \ge \frac{M_{f,Rd}}{M_{pl,Rd}} \qquad (3.53)$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \qquad (3.54)$$

2.5 Check resistance to vertical shear

$$V_{Ed} \le V_{Rd} \qquad (3.55)$$

where,
V_{Ed} *: Design value of the shear force acting on the composite section.*
V_{Rd} *: Design value of the resistance of the composite section to vertical shear.*

2.6 Verification of vertical shear resistance

By Result Table

The verification results can be checked in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Vertical Shear…

Model View | Resistance to Vertical Shear

Elem	Position	Lcom	Type	Top Class	Bot Class	Web Class	Sect. Class	N_Ed (kN)	M_Ed (kN·m)	V_Ed (kN)	Vpl,Rd (kN)	Vb,Rd (kN)
2	I[2]	sLCB1	-	1	1	2	2	-2000.0000	3184.2313	-46985.8208	48560.9168	0.0000
2	J[3]	-	-	-	-	-	-	-	-	-	-	-

Position: I/J-end
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
Top Class: Class of top flange
Bot Class: Class of bottom flange
Web Class: Class of web
Sect. Class: Class of cross section
N_Ed : Design value of the compressive normal force
M_Ed: Design bending moment
V_Ed : Design value of the shear force acting on the composite section
Vpl,Rd: Design value of the plastic resistance of the composite section to vertical shear
Vb,Rd: Design value of the shear buckling resistance of a steel web

By Excel Report

Detail results with applied equations and parameters can be checked in the Excel Report.

93	3 **Resistance to Vertical Shear**						
94	- Design load						
95	Load combination name :		sLCB1				
96	N_{Ed}	=	-2000.000	kN			
97	$M_{a,Ed}$	=	-804.929	kN · m			
98	$M_{c,Ed}$	=	-1848.218	kN · m			
99	$V_{Ed,a}$	=	-14488.720	kN			
100	$V_{Ed,c}$	=	-32497.101	kN			
101	V_{Ed}	=	-46985.821	kN			
102							
136	- Plastic resistance moment, $M_{pl,Rd}$						
137	Plastic NA	=	1343.878	mm			
138							
139	N_{slab}	=	0.000	kN			
140	$N_{g,top}$	=	73162.780	kN			
141	$N_{g,bot}$	=	81270.695	kN			
142							
143	$M_{pl,Rd}$	=	115704.166	kN · m			
144							
145	- Calculation. $V_{pl,Rd}$						
146	Web(Web_R)						
147	α	=	a/h_w	=	0.14607076		
148	k_τ	=	$4.1+(6.3+0.18 \cdot I_{sl}/(t^3 \cdot h_w)) / \alpha^2 + 2.2 \cdot (I_{sl}/(t^3 \cdot h_w))^{1/3}$	=	306.7841206		
149			I_{sl}	=	167648996.696	mm^4	
150			t	=	50.000	mm	
151							
152	$V_{pl,Rd}$	=	$Av \cdot (fy / \sqrt{(3)}) / \gamma_{M0}$	=	24308.110	kN	
153	V_{Rd}	=	24308.110	kN			
154	V_{Edi}	=	V_{Ed} / Num. of Web	=	-23492.910	kN	
155							
156	V_{Edi} / V_{Rd}	=	0.966	≤	1.0		... OK
157							
158	Interaction M-V						
159	For the section class 1 or 2, M-V interaction should be checked separately by the user.						

3. Resistance to lateral-torsional buckling

Resistance to lateral-torsional buckling is verified only for the plate girder. The following conditions must be satisfied.

$$M_{Ed} \le M_{b,Rd}$$

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{b,Rd}} \le 1.0$$

$N_{b,Rd}$, $M_{b,Rd}$ shall be calculated as follows.

3.1 Design buckling resistance moment $M_{b,Rd}$

$$M_{b,Rd} = \chi_{LT} M_{Rd} \tag{3.56}$$

EN1994-2:2005 6.4.2(1)

where,

χ_{LT} : *The reduction factor for lateral-torsional buckling corresponding to the relative slenderness* λ_{LT}

M_{Rd} : *The design resistance moment at the relevant cross-section.*

(1) The reduction factor χ_{LT}

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \le 1.0 \tag{3.57}$$

EN1993-1-1:2005 6.3.2.2

where,

$$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right] \tag{3.58}$$

[Table 3.24] Lateral torsional buckling curve for cross-section

EN1993-1-1:2005 Table 6.4

Cross section	Limits	Buckling Curve
Welded I-Section	h/b ≤ 2	c
	h/b > 2	d

In midas Civil, plate I- girder is considered as welded section. Rolled section is not considered.

α_{LT} : *An imperfection factor.*

[Table 3.25] Imperfection factor for lateral torsional buckling curves

EN1993-1-1:2005 Table 6.3

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

$$\bar{\lambda}_{LT} = 1.103\frac{L}{b}\sqrt{\frac{f_y}{Em}}\sqrt{1 + \frac{A_{wc}}{3A_f}} \tag{3.59}$$

Designers' Guide to EN1994-2, (D6.14)

L : The span length between the rigid supports.
B : The width of the compression flange.
A_{wc} : The area of the compression zone of the web.
A_f : The area of the compression flange.
$m = \min[m_1,\ m_2]$

$$m_1 = 1.0 + 0.44(1+\mu)\Phi^{1,5} + \frac{3+2\Phi}{350-50\mu} \quad (3.60)$$

$$m_2 = 1.0 + 0.44(1+\mu)\Phi^{1,5} + \left(0.195 + \left(0.05 + \frac{\mu}{100}\right)\Phi\right)\gamma^{0.5} \quad (3.61)$$

$$\gamma = \frac{cL^4}{EI} \quad (3.62)$$

$$c = \frac{C_d}{l} \quad (3.63)$$

C_d : The spring stiffness.
L : The distance between the springs.

$$\mu = \frac{V_2}{V_1}, \quad V_2 < V_1$$

[Table 3.26] Calculation of Φ

Bending moment	Φ
Change sign	$\Phi = \frac{2}{1+\mu}$
Not change sign	$\Phi = \frac{2(1-M_2/M_1)}{1+\mu}, \quad M_2 < M_1$

(2) The design resistance moment M_{Rd}

[Table 3.27] Design resistance moment for section class

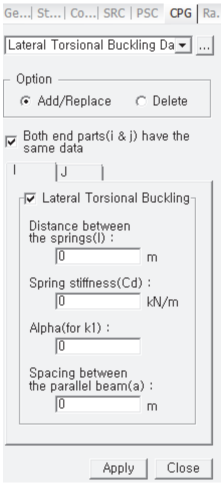
Section Class	M_{Rd}
1, 2	$M_{pl,Rd}$
3	$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed}$

In midas Civil, the verification of lateral-torsional buckling for Class 4 is done by applying the identical equation as Class 3.

Lateral torsional buckling data

Parameters required for the verification of lateral torsional buckling can be entered in the following dialog box.

Design>Composite Steel Girder Design>Lateral Torsional Buckling Data(Plate Girder Only)…



[Fig. 3.28] Lateral torsional buckling data

3.2 Axial buckling resistance of the cracked composite cross-section $N_{b,Rd}$

$$N_{b,Rd} = \chi_{LT} N_{Rd} \tag{3.64}$$

where,

χ_{LT} : *The reduction factor for lateral-torsional buckling corresponding to the relative slenderness* λ_{LT}

N_{Rd} : *The design resistance moment at the relevant cross-section.*

(1) The reduction factor χ_{LT}

The reduction factor, χ_{LT}, is calculated as per the clause 3.1. When the reduction factor due to axial force, χ_{LT}, is calculated, m=1.0 will be applied.

(2) The design resistance axial N_{Rd}

$$N_{Rd} = Af_{yd} \tag{3.65}$$

where,

A : *Cross-sectional area of the effective composite section neglecting concrete in tension.*

f_{yd} : *The design value of the yield strength of structural steel.*

3.3 Check resistance to lateral-torsional buckling

$$M_{Ed} \le M_{b,Rd} \tag{3.66}$$

$$\frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{b,Rd}} \le 1.0 \tag{3.67}$$

3.4 Verification of lateral-torsional buckling resistance

By Result Table

Verification results can be checked in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Lateral-Torsional buckling...

Model View | Resistance to Lateral-Torsional Buckling

Elem	Position	Lcom	Type	Sect. Class	N_Ed (kN)	M_Ed (kN·mm)	Nb,Rd (kN)	Mb,Rd (kN·mm)	Mcr (kN·mm)	Interaction Ratio
2	I[2]	sLCB1	-	4	-2000.0000	-1234865.3602	30499.4800	8780518.2553	115007340.8255	0.2062
2	J[3]	-	-	-	-	-	-	-	-	-

Elem: Element
Position: I/J-end
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
Sect. Class: Class of cross section
N_Ed: Design value of the compressive normal force
M_Ed: Design bending moment
Nb,Rd: Design buckling resistance of the compression member
Mb,Rd: Design buckling resistance moment
Interaction Ratio: $N_{Ed}/N_{b,Rd}+M_{Ed}/M_{b,Rd} \leq 1.0$

By Excel Report

A B C D E F G H I J K L M N O P Q R S T U V W X Y Z AA AB AC AD AE AF AG

4 **Resistance to Lateral Torsional Buckling**

- Design load

Load combination name :	sLCB1				
N_{Ed}	=	-2000.000	kN		
M_{Ed}	=	-1234.865	kN · m		
V_1	=	-25252.848	kN		
V_2	=	-8388.655	kN		
M_1	=	-1276.082	kN · m		
M_2	=	15544.669	kN · m		
$M_{pl,Rd}$	=	13217.334	kN · m		
$M_{el,Rd}$	=	8780.518	kN · m		

- $M_{b,Rd}$ Buckling resistance moment

L	=	1000.000	m		
c	=	C_d / I	=	0.100	
γ	=	$c \cdot L^4 / (E \cdot I)$	=	0.000	
μ	=	V_2 / V_1	=	0.332	
Φ	=	$2 \cdot (1-M_2/M_1) / (1+\mu)$	=	1.501	
m_1	=	$1+0.44 \cdot (1+\mu) \cdot \Phi^{1.5}+(3+2 \cdot \Phi) \cdot \gamma/(350-50 \cdot \mu)$	=	2.078	
m_2	=	$1+0.44 \cdot (1+\mu) \cdot \Phi^{1.5}+(0.195+(0.05+\mu/100) \cdot \Phi) \cdot \gamma^{0.5}$	=	2.080	
m	=	$Min(m_1, m_2)$	=	2.078	
α_{LT}	=	0.490			
λ_{LT}	=	$1.103 \cdot L/b \cdot \sqrt{(f_y/E_m)} \cdot \sqrt{(1+A_{wc}/(3 \cdot A_f))}$	=	0.056	
Φ_{LT}	=	$0.5 \cdot (1+\alpha_{LT} \cdot (\lambda_{LT} - 0.2)+\lambda_{LT}^2)$	=	0.466	
X_{LT}	=	$\frac{1}{\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \lambda_{LT}^2)}}$	=	1	
M_{Rd}	=	8780.518	kN · m		
$M_{b,Rd}$	=	$X_{LT} \cdot M_{Rd}$	=	8780.518	kN · m

- $N_{b,Rd}$ Buckling resistance moment

$X_{LT,N}$	=	1.000			
$N_{b,Rd}$	=	$X_{LT} \cdot Area \cdot f_{yd}$	=	30499.480	kN

Combined Ratio $= \frac{N_{Ed}}{N_{b,Rd}} + \frac{M_{Ed}}{M_{b,Rd}} =$ 0.206211845

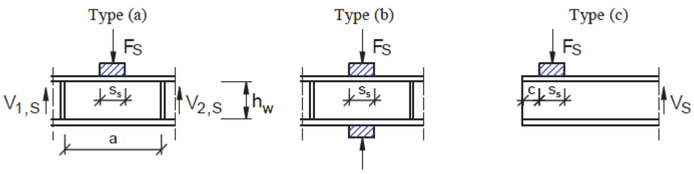
4. Resistance to transverse force

Resistance to transverse force can be verified for plate I-girder.
The following condition must be satisfied.

$$\eta_2 \le 1.0$$
$$\eta_2 + 0.8\eta_1 \le 1.4$$

η_1, η_2 shall be calculated as follows.

4.1 Type of load application



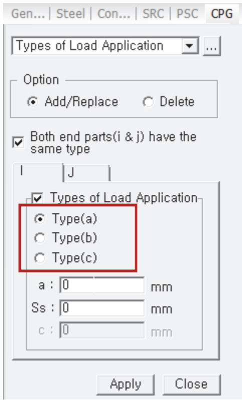
EN1993-1-5:2006 Figure 6.1

[Fig. 3.29] Buckling coefficients for different types of load application

Types of load application

Types of load application and the related parameters can be specified as follows.

Design>Composite Steel Girder Design>Type of Load Application (Plate Girder Only)...



Type of loading
(See Fig.2.25)

[Fig. 3.30] Type of load application Input Dialog

4.2 Design resistance to local buckling under transverse forces F_{Rd}

$$F_{Rd} = \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}} \qquad (3.68)$$

EN1993-1-5:2006 (6.1)

where,
f_{yw} *: The yield strength of the web.*
L_{eff} *: The effective length for resistance to transverse forces.*
t_w *: The thickness of the web.*
γ_{M1} *: The partial factor for resistance of members to instability assessed by member checks.*

(1) Effective length L_{eff}

$$L_{eff} = \chi_F l_y \tag{3.69}$$

EN1993-1-5:2006 (6.2)

where,
l_y : *The effective loaded length.*
χ_F : *The reduction factor due to local buckling.*

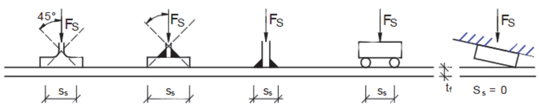
(2) Effective loaded length l_y

EN1993-1-5:2006 6.5(2),(3)

[Table 3.28] Calculation of l_y

Type of loading	l_y
(a), (b)	$l_y = s_s + 2t_f\left(1 + \sqrt{m_1 + m_2}\right)$
(c)	$l_y = \min\left[l_{y1}, l_{y2}, l_e\right]$

where,
s_s : *The length of stiff bearing.*
h_w : *Clear web depth between flanges.*



[Fig. 3.31] Length of stiff bearing

EN1993-1-5:2006 Figure 6.2

t_f : *The thickness of the flange*

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w} \tag{3.70}$$

EN1993-1-5:2006 6.5(1)

$$m_2 = \begin{cases} 0.02\left(\dfrac{h_w}{t}\right)^2 & (\overline{\lambda_F} > 0.5) \\ 0 & (\overline{\lambda_F} \le 0.5) \end{cases} \tag{3.71}$$

In midas Civil, only $m_2 = 0.02\left(\frac{h_w}{t}\right)^2$ is applied.

EN1993-1-5:2006 6.5(3)

$$l_{y1} = l_e + t_f\sqrt{\frac{m_1}{2} + \left(\frac{l_e}{t_f}\right)^2 + m_2} \tag{3.72}$$

$$l_{y2} = l_e + t_f\sqrt{m_1 + m_2} \tag{3.73}$$

$$l_e = \frac{k_F E t_w^2}{2 f_{yw} h_w} \le s_s + c \tag{3.74}$$

(3) Reduction factor for effective length for resistance χ_F

$$\chi_F = \frac{0.5}{\overline{\lambda}_F} \le 1.0 \quad (3.75)$$

EN1993-1-5:2006 6.4(1)

where,

$$\overline{\lambda}_F = \sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \quad (3.76)$$

$$F_{cr} = 0.9 k_F E \frac{t_w^3}{h_w} \quad (3.77)$$

k_F : The buckling coefficient for concentrated load.
b_1 : The depth of the loaded subpanel taken as the clear distance between the loaded flange and the stiffener.

$$\gamma_s = 10.9 \frac{I_{sl,1}}{h_w t_w^3} \le 13\left(\frac{a}{h_w}\right)^3 + 210\left(0.3 - \frac{b_1}{a}\right) \quad (3.78)$$

EN1993-1-5:2006 6.4(2)

$I_{sl,1}$: The second moments of area of the stiffener closest to the loaded flange including contributing parts of the web.

[Table 3.29] Calculation of k_F

Type of loading	Condition	k_F
(a)	$0.05 \le b_1/h_w \le 0.3$ and $b_1/a \le 0.3$	$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 + \left(5.44\frac{b_1}{a} - 0.21\right)\sqrt{\gamma_s}$
	Others	$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2$
(b)	-	$k_F = 3.5 + 2\left(\frac{h_w}{a}\right)^2$
(c)	-	$k_F = 2 + 6\left(\frac{s_s + c}{h_w}\right) \le 6$

4.3 Verification for transverse force

$$\eta_2 = \frac{F_{Ed}}{F_{Rd}} \le 1.0 \quad (3.79)$$

EN1993-1-5:2006 6.6

where,
F_{Ed} : The design transverse force.
F_{Rd} : The resistance to transverse force.

4.4 Verification for uniaxial bending

$$\eta_1 = \frac{N_{Ed}}{\frac{f_y A_{eff}}{\gamma_{M0}}} + \frac{M_{Ed} + N_{Ed} e_N}{\frac{f_y W_{eff}}{\gamma_{M0}}} \le 1.0 \quad (3.80)$$

EN1993-1-5:2006 4.6

where,
N_{Ed} : The design axial force.

M_{Ed} : The design bending moment.
e_N : The shift in the position of neutral axis.
f_y : The yield strength of girder.
A_{eff} : The effective cross-section area.
W_{eff} : The effective elastic section modulus.
γ_{M0} : The partial factor.

4.5 Check resistance to transverse force

EN1993-1-5:2006 6.6 and 7.2

$$\eta_2 \le 1.0 \tag{3.81}$$

$$\eta_2 + 0.8\eta_1 \le 1.4 \tag{3.82}$$

4.6 Verification of transverse force resistance

By Result Table

Verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Transverse Force...

Model View | Resistance to Transverse Force

Elem	Position	Lcom	Type	F_Ed (kN)	N_Ed (kN)	My,Ed (kN·mm)	Mz,Ed (kN·mm)	F_Rd (kN)	Eta2	Eta1	Interaction Ratio
2	I[2]	sLCB1	-	-25252.8476	-2000.0000	-1276081.7352	0.0000	9279.8606	2.7213	0.1162	2.8142
2	J[3]	-	-	-	-	-	-	-	-	-	-

Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
F_Ed: Design transverse force
N_Ed: Design value of the compressive normal force
My,Ed: Design bending moment applied to the composite section about the y-y axis
Mz,Ed: Design bending moment applied to the composite section about the z-z axis
F_Rd: Design resistance to local buckling under transverse forces
Eta2: $F_{Ed}/F_{Rd} \le 1.0$
Eta1: Member verification for uniaxial bending (EN 1993-1-5, (4.14))
Interaction Ratio: $\eta_2+0.8\eta_1 \le 1.4$

By Excel Report

5 Stress Limitation						
- In the structural steel						
Characteristic load combination name :	sLCB2					
$\sigma_{Ed,ser}$ =	30.087	MPa	(Bottom-right fiber in the flange)			
$\tau_{Ed,ser}$ =	158.409	MPa	(Neutral axis in the web)			
$\sigma_{Ed,ser}$		$\le$	f_y / $\gamma_{M,ser}$			
30.087	MPa	$\le$	440.000	MPa	...	OK
$\tau_{Ed,ser}$		$\le$	f_y / ($\sqrt{3}$ · $\gamma_{M,ser}$)			
158.409	MPa	$\le$	254.034	MPa	...	OK
$\sqrt{(\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2)}$		$\le$	f_y / $\gamma_{M,ser}$			
276.018	MPa	$\le$	440.000	MPa	...	OK
- In the concrete of the slab						
Characteristic load combination name :	sLCB2					
$\sigma_c \le k_1 f_{ck}$						
0.000	MPa	$\le$	24.000	MPa	...	OK
- In the reinforcement						
Load combination name :	sLCB2					
$\sigma_s \le k_3 f_{sk}$						
6.564	MPa	$\le$	320.000	MPa	...	OK

5. Resistance to longitudinal shear

Resistance to longitudinal shear is verified only for the plate I-girder and the following condition must be satisfied.

$$v_{L,Ed} \leq v_{L,Rd}$$

$v_{L,Ed}$, $v_{L,Rd}$ shall be calculated as follows.

5.1 Design shear resistance of headed stud

EN1994-2:2005 6.6.3.1(1)

$$P_{Rd} = \min[P_{Rd1}, P_{Rd2}] \quad (3.83)$$

$$P_{Rd1} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} \quad (3.84)$$

$$P_{Rd2} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \quad (3.85)$$

where,
γ_V *: The partial factor.*
d *: The diameter of the shank of the stud.*
f_u *: The specified ultimate tensile strength of the material of the stud.*
f_{ck} *: The characteristic cylinder compressive strength of the concrete at the age considered.*
h_{sc} *: The overall nominal height of the stud.*

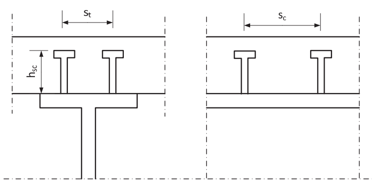
[Table 3.30] Calculation of α

EN1994-2:2005 (6.20),(6.21)

$3 \leq h_{sc}/d \leq 4$	$h_{sc}/d > 4$
$\alpha = 0.2\left(\frac{h_{sc}}{d} + 1\right)$	α =1

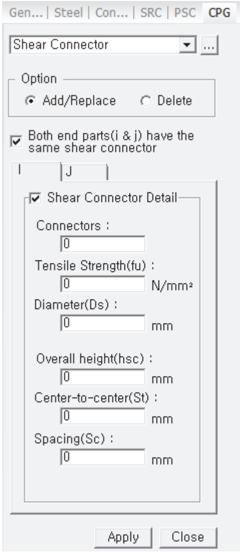
Shear connector

For shear connectors, enter the number of connectors, tensile strength, dimension, height (h_{sc}), transverse spacing (s_t), and longitudinal spacing (s_c).



[Fig. 3.32] Notation of shear connector

Design>Composite Steel Girder Design>Shear Connector…



[Fig. 3.33] Shear connector Input Dialog

5.2 Bearing shear stress of shear connector, $v_{L,Rd}$

$$v_{L,Rd} = \frac{P_{Rd}N}{s_c} \tag{3.86}$$

EN 1994-2:2005
6.6.2.2

where,
N : The number of the shear connector.
s_c : The space of the shear connector.

5.3 Shear stress at the connection between girder and deck, $v_{L,Ed}$

(1) Beams with cross-sections in Class 1 or 2 and under the sagging moment and inelastic behavior ($M_{Ed} > M_{el,Rd}$).

$$v_{L,Ed} = \frac{V_{L,Ed}}{L_v} \tag{3.87}$$

where,

$$V_{L,Ed} = \frac{\left(N_{c,f} - N_{c,el}\right)\left(M_{ED} - M_{el,Rd}\right)}{M_{pl,Rd} - M_{el,Rd}} \tag{3.88}$$

L_v : Length of shear connection. ($L_v = b_{eff} = B_c$)

(2) Other cases

$$v_{L,Ed} = \frac{V_{Ed}Q_s}{I_y} \tag{3.89}$$

where,
Q_s : Geometric moment of area at the shear connector position (contact point between girder and slab)

[Table 3.31] Calculation of Q_s

Condition	Q_s
Gravity center of composite section < Height of girder	Calculate the geometric moment of area with slab
Gravity center of composite section ≥ Height of girder	Calculate the geometric moment of area with girder

5.4 Check resistance to longitudinal shear

$$v_{L,Ed} \leq v_{L,Rd} \tag{3.90}$$

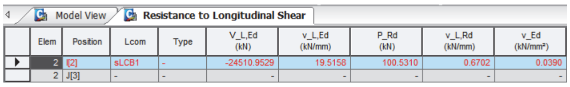
where,
$v_{L,Ed}$ *: Design longitudinal shear force per unit length at the interface between steel and concrete.*
$v_{L,Rd}$ *: Resistance to longitudinal shear.*

5.5 Verification of longitudinal shear resistance

By Result Table

Verification results can be checked as shown in the table below.

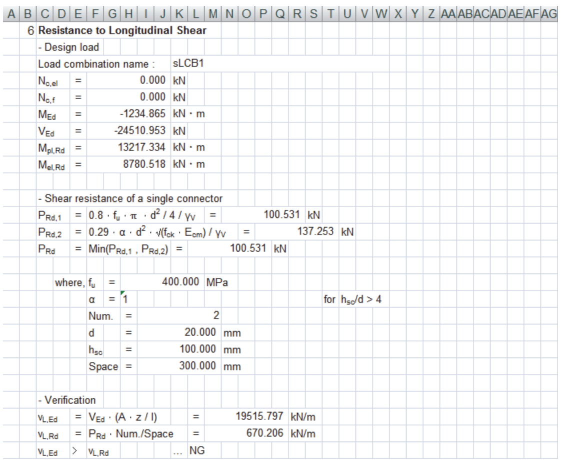
Design>Composite Steel Girder Design>Design Result Tables>Resistance to Longitudinal Shear...


Model View | Resistance to Longitudinal Shear

Elem	Position	Lcom	Type	V_L,Ed (kN)	v_L,Ed (kN/mm)	P_Rd (kN)	v_L,Rd (kN/mm)	v_Ed (kN/mm²)
2	I[2]	sLCB1	-	-24510.9529	19.5158	100.5310	0.6702	0.0390
2	J[3]	-	-	-	-	-	-	-

Elem: Element
Position: I/J-end
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
V_L,Ed: Longitudinal shear force acting on length of the inelastic region
v_L,Ed: Design longitudinal shear force per unit length at the interface between steel and concrete
P_Rd: Design value of the shear resistance of a single connector
v_Ed: Design longitudinal shear stress

By Excel Report


6 Resistance to Longitudinal Shear
- Design load
Load combination name : sLCB1
$N_{c,el}$ = 0.000 kN
$N_{c,f}$ = 0.000 kN
M_{Ed} = -1234.865 kN · m
V_{Ed} = -24510.953 kN
$M_{pl,Rd}$ = 13217.334 kN · m
$M_{el,Rd}$ = 8780.518 kN · m

- Shear resistance of a single connector
$P_{Rd,1}$ = $0.8 \cdot f_u \cdot \pi \cdot d^2 / 4 / \gamma_V$ = 100.531 kN
$P_{Rd,2}$ = $0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{(f_{ck} \cdot E_{cm})} / \gamma_V$ = 137.253 kN
P_{Rd} = $Min(P_{Rd,1}, P_{Rd,2})$ = 100.531 kN

where, f_u = 400.000 MPa
α = 1 for $h_{sc}/d > 4$
Num. = 2
d = 20.000 mm
h_{sc} = 100.000 mm
Space = 300.000 mm

- Verification
$v_{L,Ed}$ = $V_{Ed} \cdot (A \cdot z / I)$ = 19515.797 kN/m
$v_{L,Rd}$ = $P_{Rd} \cdot$ Num./Space = 670.206 kN/m
$v_{L,Ed} > v_{L,Rd}$ ... NG

6. Resistance to fatigue

Resistance to fatigue should satisfy the following condition.

$$\gamma_{Ff}\Delta\tau_{E,2} \le \frac{\Delta\tau_c}{\gamma_{Mf,s}}$$

$\Delta\tau_{E,2}, \Delta\tau_c$ will be calculated as follows.

6.1 Partial factors for fatigue

(1) Partial factor for fatigue resistance γ_{Mf}

EN1993-1-9:2005 Table 3.1

[Table 3.32] Recommended values for partial factor

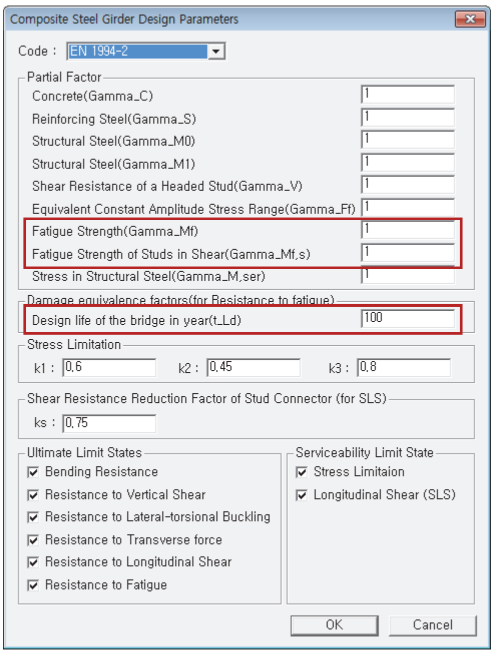
Assessment Method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1.00	1.15
Safe life	1.15	1.35

(2) Partial factor for fatigue loads γ_{Ff}

Recommend value = 1.0

Design parameters for fatigue

Partial factor and design life of the bridge in year can be entered in Composite Steel Girder Design Parameters dialog box.



[Fig. 3.34] Composite Steel Girder Design Parameters

6.2 Equivalent constant range of shear stress for 2million cycles $\Delta\tau_{E,2}$

$$\Delta\tau_{E,2} = \lambda_v \Delta\tau \quad (3.91)$$

EN1994-2:2005 6.8.6.2(1)

where,

λ_v *:The damage equivalent factor depending on the spectra and the slope m of the fatigue strength curve.*

$\Delta\tau$ *: The range of shear stress due to fatigue loading.*

(1) Damage equivalent factor λ_v

$$\lambda_v = \lambda_{v,1}\lambda_{v,2}\lambda_{v,3}\lambda_{v,4} \quad (3.92)$$

EN1994-2:2005 6.8.6.2(3)~(5)

where,

$\lambda_{v,1}$ *: The factor for the damage effect of traffic and depends on the length of the critical influence line or area.*

$\lambda_{v,2}$ *: The factor for the traffic volume.*

$\lambda_{v,3}$ *: The factor for the design life of the bridge.*

$$\lambda_{v,3} = \left(\frac{t_{Ld}}{100}\right)^{1/5} \quad (3.93)$$

EN1993-2:2006 9.5.2(5)

t_{Ld} *: The design life of the bridge in years.*

$\lambda_{v,4}$ *: The factor for the traffic on other lanes.*

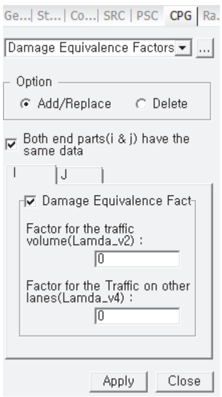
In midas Civil $\lambda_{v,1}$ is applied as "1.55" and t_{Ld} for calculating $\lambda_{v,2}$, $\lambda_{v,4}$, and $\lambda_{v,3}$ can be entered by the user. Refer to the clause 6.1 for the input parameter.

(2) Range of shear stress $\Delta\tau$

Calculate the shear stress per a shear connector.

Damage equivalent factor

The values for $\lambda_{v,2}$ and $\lambda_{v,4}$ can be specified by the members as shown in the figure below.



[Fig. 3.35] Damage Equivalence Factors

6.3 Reference value of fatigue strength at 2 million cycles $\Delta\tau_c$

The value of $\Delta\tau_c$ is applied as 90 N/mm^2.

EN1994-2:2005 6.8.3(3)

6.4 Check resistance to fatigue

$$\gamma_{Ff}\Delta\tau_{E,2} \leq \frac{\Delta\tau_c}{\gamma_{Mf,s}} \tag{3.94}$$

EN1994-2:2005 6.8.7.2

where,

γ_{Ff} *: The partial factor for fatigue loading.*

$\gamma_{Mf,s}$ *: The partial factor for head studs in shear.*

6.5 Verification of fatigue resistance

By Result Table

The verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Resistance to Fatigue...

Model View | **Resistance to Fatigue**

Elem	Position	Lcom	Type	lamda_v	delta Tau (kN/mm²)	delta Tau_E,2 (kN/mm²)	delta Tau_c (kN/mm²)	Ratio
2	I[2]	sLCB1	-	0.0000	9.3181	0.0000	0.0900	0.0000
2	J[3]	-	-	-	-	-	-	-

Elem: Element
Position: I/J-end
Lcom: Load combination
Type: Load combination type (Fxx-max, Fxx-min, ... Mzz-min)
lamda_v: Damage equivalent factors
delta Tau: Range of shear stress for fatigue loading
delta Tau_E,2: Equivalent constant amplitude range of shear stress related to 2 million cycles
delta Tau_c: Reference value of the fatigue strength at 2 million cycles
Ratio: delta Tau_E,2/ delta Tau_c

By Excel Report

7 **Resistance to Fatigue**

- Design load

Load coml sLCB1

M_y = -1276.082 kN · m

- Shear stress range for the connector

$\Delta\tau$ = F_{sc} / A_{sc} = 9318.107 MPa

where, F_{sc} = $v_{L,Ed}$ · space of stud / number of stud = 2927.370 kN

A_{sc} = 314.159 mm^2

- Damage equivalent factor

λ_v = $\lambda_{v,1} \cdot \lambda_{v,2} \cdot \lambda_{v,3} \cdot \lambda_{v,4}$ = 0.000

where, $\lambda_{v,1}$ = 1.550

$\lambda_{v,2}$ = 0.000

$\lambda_{v,3}$ = 1.000

$\lambda_{v,4}$ = 0.000

- Equivalent constant amplitude range of shear stress related to 2 million cycles

$\Delta\tau_{E,2}$ = $\lambda_v \cdot \Delta\tau$ = 0.000 MPa

- Verification

$\gamma_{Ff} \cdot \Delta\tau_{E,2}$ / ($\Delta\tau_c$ / γ_{Mfs}) = 0.000 ≤ 1

Serviceability Limit States

1. Stress limitation

For the stress limit check of plate girder, the following stress will be calculated and compared to its allowable stress: Normal stress of girders, Shear stress of girders, Combined stress of girders, stress in slab, and stress in rebar. Each stress can be calculated as follows.

1.1 Stress limitation for girder

(1) Normal stress $\sigma_{Ed,ser}$

$$\sigma_{Ed,ser} \le \sigma_{allow} = \frac{f_y}{\gamma_{M,ser}} \quad (3.95)$$

EN1994-2:2005 7.2.2(5)

EN1993-2:2006 (7.1)

• Stress in girder, $\sigma_{Ed,ser}$, is calculated by the stresses summation of before composite and after composite state at 4 different points. Member forces and section properties are calculated as shown in the table below.

[Table 3.33] Member forces for calculating girder stress

Type	Before composite	After composite
Section Properties	Girder	Sagging moment: Deck concrete + Girder Hogging moment: Deck rebar + Girder
Member Force	Calculate using girder only	Calculate considering deck concrete and girder

In midas Civil, applied section properties can be verified in the excel report. The section properties of before composite action is shown as “Before”, after composite action is shown as “After”, negative moment with considering cracked section is shown as “Crack”.

(2) Shear stress $\tau_{Ed,ser}$

$$\tau_{Ed,ser} \le \tau_{allow} = \frac{f_y}{\sqrt{3}\gamma_{M,ser}} \quad (3.96)$$

EN1993-2:2006 (7.2)

where,

$$\tau_{Ed,ser} = \frac{V_{Ed}}{A_v} \quad (3.97)$$

V_{Ed} *: Shear force after composite action*
A_v *: Shear area. For I-girder,* $A_v = h_w t_w$. *For the other sections,* $A_v = \sum A_{web}$.

(3) Combined stress $\sigma_{Ed,com,ser}$

$$\sigma_{Ed,com,ser} \le \sigma_{allow} = \frac{f_y}{\gamma_{M,ser}} \tag{3.98}$$

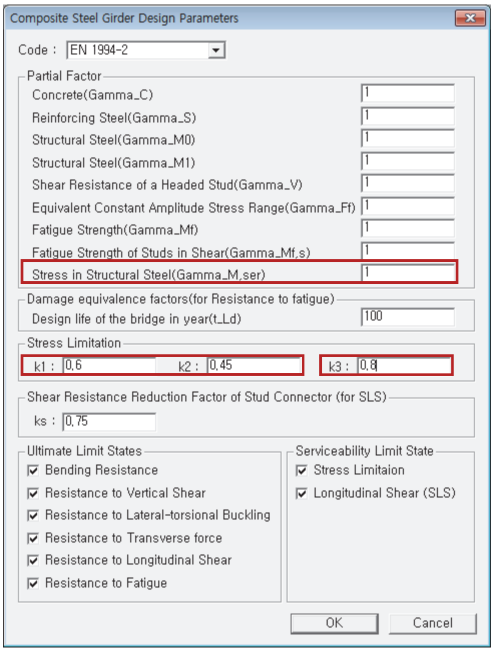
where,

$$\sigma_{Ed,com,ser} = \sqrt{\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2} \tag{3.99}$$

EN1993-2:2006
(7.3)

Stress limitation parameters

Design > Composite Steel Girder Design > Design Parameters...



$\gamma_{M,ser}$

k_1, k_2, k_3

[Fig. 3.36] Composite Girder Design Parameters

1.2 Stress limitation for concrete of slab

$$\sigma_c \le \sigma_{allow} = kf_{ck} \tag{3.100}$$

where,

k : It is used as the user defined value. Refer to 3.1.1.1 for the input parameter of k_1, k_2.

[Table 3.34] Recommended value of k for concrete

Serviceability Load combination Type	k	
	Applied	Recommended
Characteristic	k_1	0.6
Quasi-permanent	k_2	0.45

EN1994-2:2005
7.2.2(2)

f_{ck} : The characteristic value of the cylinder compressive strength of concrete at 28 days.

1.3 Stress limitation for reinforcement of slab

$$\sigma_s \le \sigma_{allow} = k_3 f_{sk} \qquad (3.101)$$

where,

k_3 *: It is used as the user defined value.*

EN1994-2:2005 7.2.2(4)

[Table 3.35] Recommended value of k for reinforcement

Serviceability Load combination Type	k	
	Applied	Recommended
Characteristic	k_3	0.45

f_{sk} *: Characteristic value of the yield strength of reinforcing steel.*

1.4 Verification of stress limitation resistance

By Result Table

The verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Stress Limitation…

Model View | Stress Limitation

Elem	Position	Top and Bottom Flange of Structural Steel								Concrete Deck				Reinforcement in Deck			
		Lcom	Type	Sigma_Ed,ser (N/mm²)	ALW (N/mm²)	Tau_Ed,ser (N/mm²)	ALW (N/mm²)	SQRT(sigma^2+3tau^2) (N/mm²)	ALW (N/mm²)	Lcom	Type	Sigma_c (N/mm²)	k*fck (N/mm²)	Lcom	Type	Sigma_s (N/mm²)	k*fsk (N/mm²)
2	I[2]	sLCB2	Characteris	30.0868	440.0000	158.4094	254.0341	276.0178	440.0000	sLCB2	Characteris	-0.0000	24.0000	sLCB2	Characteri	6.5638	320.0000
2	J[3]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Sigma_Ed,ser, Tau_Ed,ser: Nominal stresses in the structural steel from the characteristic load combination. Refer to EN 1993-2 7.3.
ALW: Stress limit.
Sigma_c: Stress in the concrete deck.
*k*fck: Stress limit.*
Sigma_s: stress in the reinforcement.
*k*fsk: stress limit.*

By Excel Report

8 **Stress Limitation**

- In the structural steel

Characteristic load combination name : sLCB3

$\sigma_{Ed,ser}$ = 65.343 MPa (Bottom-right fiber in the flange)

$\tau_{Ed,ser}$ = 612.899 MPa (Neutral axis in the web)

$\sigma_{Ed,ser} \le f_y / \gamma_{M,ser}$

65.343 MPa ≤ 440.000 MPa ... OK

$\tau_{Ed,ser} \le f_y / (\sqrt{3} \cdot \gamma_{M,ser})$

612.899 MPa > 254.034 MPa ... NG

$\sqrt{(\sigma_{Ed,ser}^2 + 3\tau_{Ed,ser}^2)} \le f_y / \gamma_{M,ser}$

1063.582 MPa > 440.000 MPa ... NG

- In the concrete of the slab

Quasi-permanent load combination name : sLCB2

$\sigma_c \le k_2 f_{ck}$

0.000 MPa ≤ 18.000 MPa ... OK

- In the reinforcement

Load combination name : sLCB3

$\sigma_s \le k_3 f_{sk}$

19.995 MPa ≤ 320.000 MPa ... OK

2. Longitudinal shear in SLS (Serviceability Limit States)

Resistance to longitudinal shear can be verified for the I-girder and following condition must be satisfied.

$$v_{L,Ed} \le v_{L,Rd}$$

$v_{L,Ed}$, $v_{L,Rd}$ shall be calculated as follows.

2.1 Design shear resistance of headed stud

EN1994-2:2005 6.6.3.1(1)

$$P_{Rd} = \min[P_{Rd1}, P_{Rd2}] \quad (3.102)$$

$$P_{Rd1} = \frac{0.8 f_u \pi d^2 / 4}{\gamma_V} \quad (3.103)$$

$$P_{Rd2} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} \quad (3.104)$$

where,
γ_V : *The partial factor.*
d : *The diameter of the shank of the stud,* $16mm \le d \le 25mm$.
f_u : *The specified ultimate tensile strength of the material of the stud,* $\le 500N/mm^2$.
f_{ck} : *The characteristic cylinder compressive strength of the concrete at the age considered.*
h_{sc} : *The overall nominal height of the stud.*
α : *Refer to Table 3.30.*

EN1994-2:2005 (6.20),(6.21)

Shear connector parameters

Shear connector is entered by members. Refer to Fig. 3.33 for the input method.

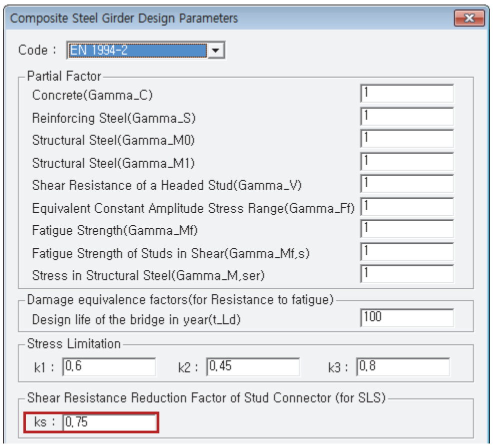
2.2 Bearing shear stress of shear connector, $v_{L,Rd}$

$$v_{L,Rd} = \frac{k_s P_{Rd} N_{conn}}{s_{conn}} \quad (3.105)$$

where,
k_s : *Reduction factor for shear resistance of stud connector.*
N_{conn} : *The number of the shear connector.*
s_{conn} : *The space of the shear connector.*

Reduction factor k_s

Reduction factor for stud, k_s, can be entered in Composite Steel Girder Design Parameters dialog box.



[Fig. 3.37] Composite Girder Design Parameters

2.3 Shear stress at the connection between girder and deck, $v_{L,Ed}$

(1) Beams with cross-sections in Class 1 or 2 and under the sagging moment and inelastic behavior ($M_{Ed} > M_{el,Rd}$)

$$v_{L,Ed} = \frac{V_{L,Ed}}{L_v} \tag{3.106}$$

where,

$$V_{L,Ed} = \frac{\left(N_{c,f} - N_{c,el}\right)\left(M_{ED} - M_{el,Rd}\right)}{M_{pl,Rd} - M_{el,Rd}} \tag{3.107}$$

EN1994-2:2005
6.6.2.2

L_v : Length of shear connection. ($L_v = b_{eff} = B_c$)

(2) Other cases

$$v_{L,Ed} = \frac{V_{Ed} Q_s}{I_y} \tag{3.108}$$

where,
Q_s : Geometric moment of area at the shear connector position (contact point between girder and slab). Refer to Table 3.31 to see the calculation method.

2.4 Check resistance to longitudinal shear in SLS

$$v_{L,Ed} \le v_{L,Rd} \tag{3.109}$$

where,
$v_{L,Ed}$: Design longitudinal shear force per unit length at the interface between steel and concrete.
$v_{L,Rd}$: Resistance to longitudinal shear.

2.5 Verification of longitudinal shear in SLS

By Result Table

Verification results can be checked as shown in the table below.

Design>Composite Steel Girder Design>Design Result Tables>Longitudinal Shear in SLS…

Model View | **Longitudinal Shear(SLS)**

Elem	Position	Lcom	Type	V_c,Ed (kN)	v_L,Ed (kN/mm)	P_Rd_ser (kN)	v_L,Rd (kN/mm)
2	I[2]	sLCB3	Characteristic	-21604.7029	17.2018	75.3982	0.5027
2	J[3]	-	-	-	-	-	-

V_c,Ed: Vertical shear force acting on the composite section.
v_L,Ed: Longitudinal shear force per unit length in the shear connector.
P_Rd_ser: Shear resistance of a single shear connector for SLS.
v_L,Rd: Longitudinal shear resistance per unit length for the shear connector.

By Excel Report

9 Longitudinal Shear for SLS(Serviceability limit state)

- Shear resistance of a single connector

Load combination name : sLCB3

$P_{Rd,1}$	=	$0.8 \cdot f_u \cdot \pi \cdot d^2 / 4 / \gamma_V$	=	100.531	kN
$P_{Rd,2}$	=	$0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{(f_{ck} \cdot E_{cm})} / \gamma_V$	=	137.253	kN
P_{Rd}	=	$\mathrm{Min}(P_{Rd,1}, P_{Rd,2})$	=	100.531	kN
$P_{Rd,ser}$	=	$k_s \cdot P_{Rd}$	=	75.398	kN

where, f_u	=	400.000	MPa	
α	=	1		for $h_{sc}/d > 4$
Num.	=	2		
d	=	20.000	mm	
h_{sc}	=	100.000	mm	
Space	=	300.000	mm	
k_s	=	0.750		

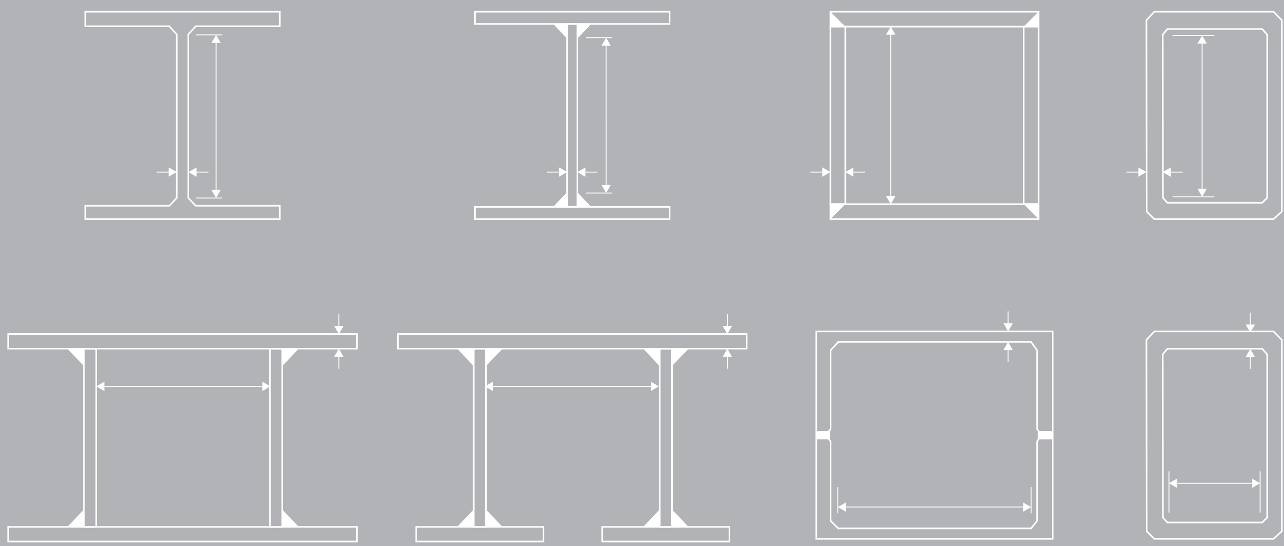
- Verification

$v_{L,Ed}$	=	$V_{Ed} \cdot (A \cdot z / I)$	=	17201.820	kN/m
$v_{L,Rd}$	=	$P_{Rd,ser} \cdot$ Num./Space	=	502.655	kN/m
$v_{L,Ed}$	>	$v_{L,Rd}$	…	NG	

Chapter 4.

Steel
Frame Design

EN 1993-2



Chapter 4.

Steel Frame Design (EN 1993-2)

Steel girder and column need to be designed to satisfy the following limit states.

Classification of Cross Section

Ultimate Limit States

- Torsion
- Compression
- Bending Moment
- Shear
- Bending and Shear
- Bending and Axial Force
- Bending, Shear, and Axial Force
- Buckling Resistance of Members

Serviceability Limit States

- Vertical Deflection
- Horizontal Deflection

Classification of Cross Section

1. Classification of cross sections

For classes of cross-sections are defined in EN1993-1-1:2005, 5.5.2 as follows:

[Table 4.1] Classes of cross-sections

Class	Definition
1	which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance
2	which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling
3	in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance
4	in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section

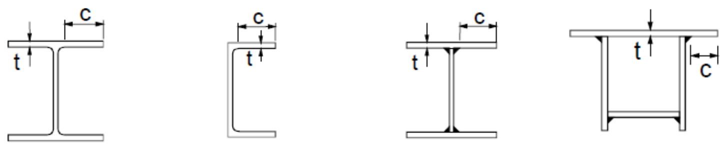
A cross-section is classified according to the highest (least favorable) class of its compression parts as following table.

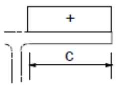
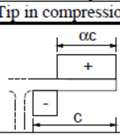
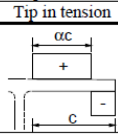
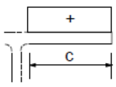
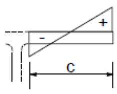
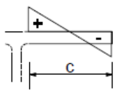
[Table 4.2] Class of section according to class of compression parts

Class of Section		Class of Flange			
		1	2	3	4
Class of Web	1	1	2	3	4
	2	1	2	3	4
	3	3	3	3	4
	4	4	4	4	4

EN1993-1-1:2005
5.5.2

Outstand flanges



Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

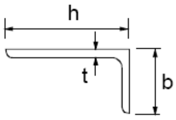
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

[Fig. 4.1] Maximum width-to-thickness ratios for compression parts – Outstand

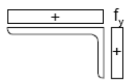
EN1993-1-1:2005
Table 5.2

Angles

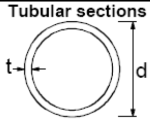
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \dfrac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

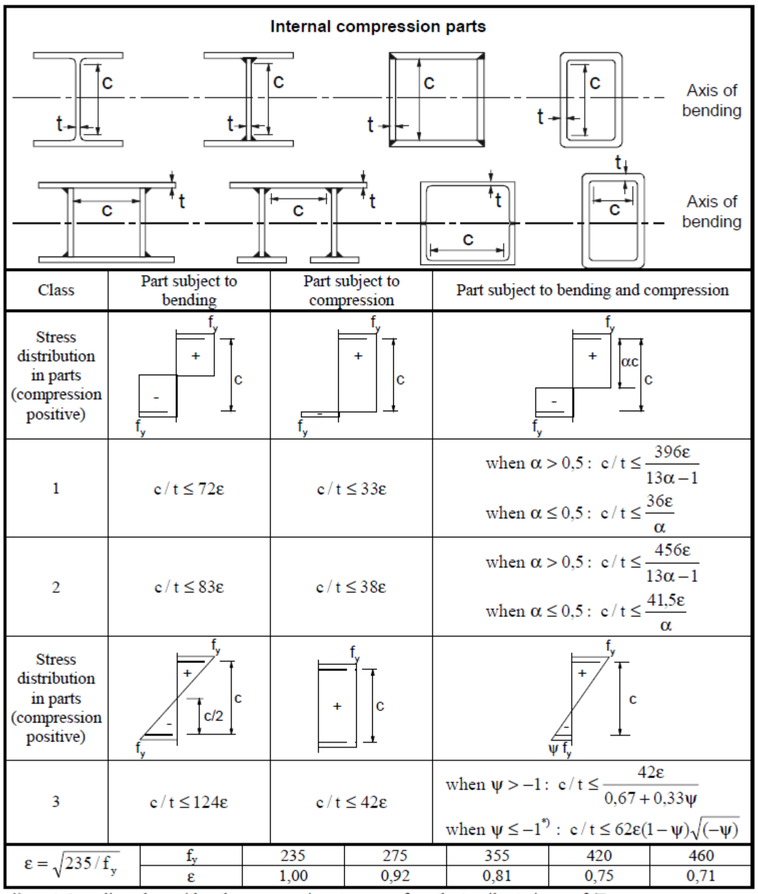
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

[Fig. 4.2] Maximum width-to-thickness ratios for compression parts - Outstand

EN1993-1-1:2005
Table 5.2

EN1993-1-1:2005
Table 5.2

• Classification of web: Check for internal compression part in Figure 1.3.



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)	f_y, +, -, f_y, c	f_y, +, -, f_y, c	f_y, +, -, f_y, αc, c
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)	f_y, +, -, f_y, c/2, c	f_y, +, c	f_y, +, -, ψf_y, c
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma < f_y$ or the tensile strain $\varepsilon_y > f_y/E$

[Fig.4.3] Maximum width-to-thickness ratios for compression parts - Internal

[Table 4.3] Section types which are not provided in Eurocode specification

Section	Element	Ratio Checked	Class 1	Class 2	Class 3
T Section	Web	h/t (b+h)/2t	Not applicable	Not applicable	15ε 11,5ε
	Flange	c/t	Same as I shape web	Same as I shape web	Same as I shape web
Inverted	Web	h/t (b+h)/2t	Not applicable	Not applicable	15ε 11,5ε
	Flange	c/t	Same as I shape web	Same as I shape web	Same as I shape web

2. Calculate effective cross-section for Class 4 section

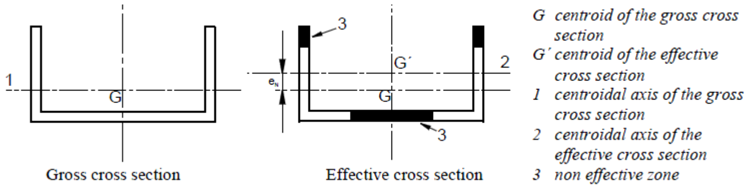
(1) Calculate effective cross-section

For cross-sections in Class4, the effective structural steel section should be determined in accordance with EN1993-1-5, 4.3.

In midas Civil, effective cross-section is determined by considering plate buckling effect without shear lag effect.

EN1993-1-5:2004
Figure 4.1

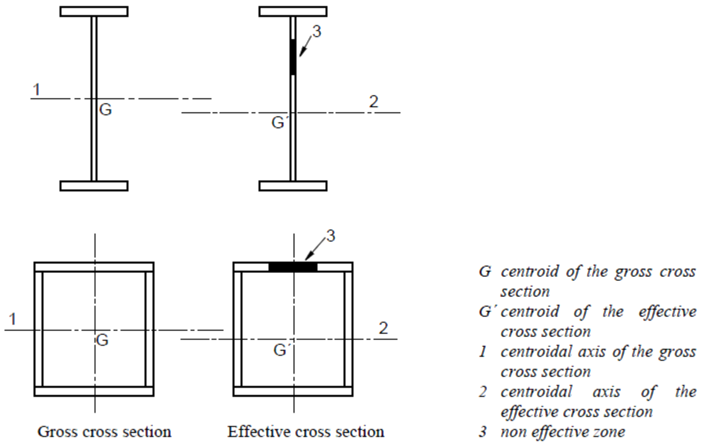
• The effective area A_{eff} should be determined assuming that the cross section is subject only to stresses due to uniform axial compression.



[Fig.4.4] Class 4 cross-sections - axial force

EN1993-1-5:2004
Figure 4.2

• The effective section modulus W_{eff} should be determined assuming that the cross section is subject only to bending stresses.



[Fig.4.5] Class 4 cross-sections - bending moment

EN1993-1-1:2005
6.2.2.5(4)

(2) Additional moment due to eccentricity of center of effective section under compression

In the section Class 4, additional moment due to changes of centroid between gross section and effective section is added in design moment.

$$\Delta M_{Ed} = N_{Ed} e_N = N_{Ed}\left(C_{z,c} - C_{z,c,eff}\right) \tag{4.1}$$

where,

e_N *: Eccentricity between centroid of gross section and centroid of effective section*

$C_{z,c}$ *: Centroid of Gross Section*

$C_{z,c,eff}$ *: Centroid of Effective Section*

3. Plate elements without longitudinal stiffeners

The effective areas of flat compression elements should be obtained using Table 4.4 for internal elements and Table 4.5 for outstand elements. The effective area of the compression zone of plate should be obtained from :

EN1993-1-5:2004
4.4

$$A_{c,eff} = \rho A_c \quad (4.2)$$

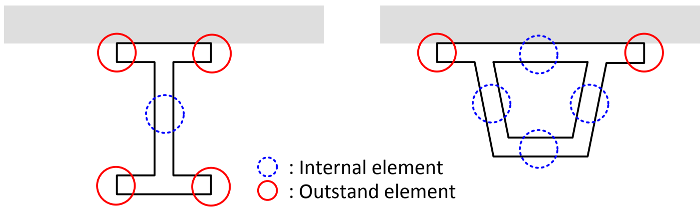
where,
$A_{c,eff}$: effective cross sectional area
A_c : the gross cross sectional area
ρ : the reduction factor for plate buckling

(1) Effective width beff

Internal element and outstand element are determined as shown in the table below.

[Table 4.4] Definition of internal and outstand element

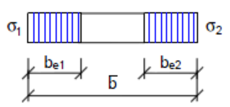
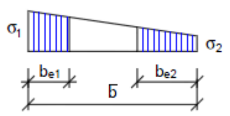
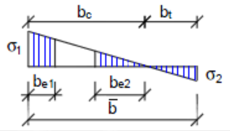
Type	Shape	Defined as
Internal element	I	Web
	Box	Web / Flanges between web
Outstand element	I	Flange
	Box	Outside parts of flange with referring to the web position



[Fig.4.6] Internal and outstand element

• For internal compression elements

EN1993-1-5:2004
Table 4.1

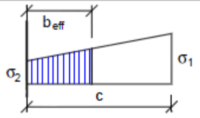
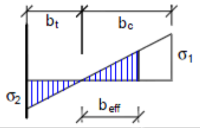
Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\psi = 1$: $b_{eff} = \rho\ \bar{b}$; $b_{e1} = 0{,}5\ b_{eff}$; $b_{e2} = 0{,}5\ b_{eff}$
	$1 > \psi \geq 0$: $b_{eff} = \rho\ \bar{b}$; $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$; $b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$; $b_{e1} = 0{,}4\ b_{eff}$; $b_{e2} = 0{,}6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\ (1 - \psi)^2$

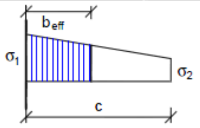
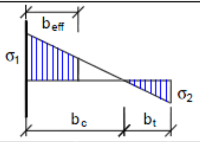
[Fig. 4.7] Internal compression elements

EN1993-1-5:2004
Table 4.2

• For outstand compression elements

<table>
<tr><th>Stress distribution (compression positive)</th><th>Effectivep width b_{eff}</th></tr>
<tr><td>
</td><td>$\underline{1 > \psi \geq 0}$:
$b_{eff} = \rho\, c$</td></tr>
<tr><td>
</td><td>$\underline{\psi < 0}$:
$b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$</td></tr>
</table>

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$

<table>
<tr><td>
</td><td>$\underline{1 > \psi \geq 0}$:
$b_{eff} = \rho\, c$</td></tr>
<tr><td>
</td><td>$\underline{\psi < 0}$:
$b_{eff} = \rho\, b_c = \rho\, c\,/\,(1-\psi)$</td></tr>
</table>

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578\,/\,(\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,1\psi^2$	23,8

[Fig. 4.8] Outstand compression elements

(2) Reduction factor ρ

EN1993-1-5:2004
4.4(2)

[Table 4.5] Calculation of reduction factor ρ

Type	Condition	ρ
Internal element	$\overline{\lambda}_p \leq 0.673$	1.0
	$\overline{\lambda}_p > 0.673$ where, $(3+\psi) \geq 0$	$\dfrac{\overline{\lambda}_p - 0.055(3+\psi)}{\overline{\lambda}_p^{\,2}} \leq 1.0$
Outstand element	$\overline{\lambda}_p \leq 0.748$	1.0
	$\overline{\lambda}_p > 0.748$	$\dfrac{\overline{\lambda}_p - 0.188}{\overline{\lambda}_p^{\,2}} \leq 1.0$

where,

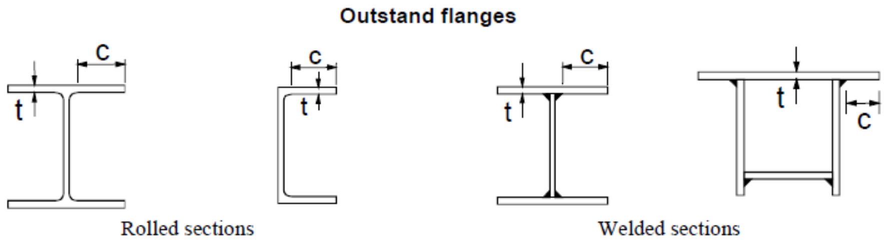
$$\overline{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\overline{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} \qquad (4.3)$$

$\overline{b}$ *: the appropriate width to be taken as follow.*

b_w *: for webs*

b : for internal flange elements.

c : for outstand flanges.



[Fig. 4.9] Dimension of outstand flanges

Ψ *: the stress ratio.*
k_σ *: the buckling factor corresponding to the stress ratio* ψ *and boundary conditions.*
t *: the thickness.*
σ_{cr} *: the elastic critical plate buckling stress.*

$$\varepsilon = \sqrt{\frac{235}{f_y[\mathrm{N/mm^2}]}} \tag{4.4}$$

Ultimate Limit States

1. Tension

EN1993-1-1:2005 6.2.3

The design value of the tension force N_{Ed} at each cross section shall satisfy:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{4.5}$$

For sections with holes the design tension resistance Nt,Rd should be taken as the smaller of

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) \tag{4.6}$$

1.1 Design plastic resistance of the gross cross-section

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{4.7}$$

1.2 Design ultimate resistance of the net cross-section for fasteners

$$N_{u,Rd} = \frac{0.9A_{net} f_u}{\gamma_{M2}} \tag{4.8}$$

midas Civil does not consider fastener holes.

2. Compression

EN1993-1-1:2005 6.2.4

The design value of the compression force N_{Ed} at each cross section shall satisfy:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \tag{4.9}$$

2.1 Design resistance of the cross-section for uniform compression Nc,Rd

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \quad \text{For class 1, 2 or 3 cross-sections} \tag{4.10}$$

$$N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} \quad \text{For class 4 cross-sections} \tag{4.11}$$

3. Bending moment

The design value of the bending moment M_{Ed} at each cross section shall satisfy:

EN1993-1-1:2005 6.2.5

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \qquad (4.12)$$

where, $M_{c,Rd}$ is determined considering fastener holes, see EN 1993-1-1:2005 (4) to (6). midas Civil does not consider fastener holes.

(1) The design resistance for bending about one principal axis of a cross-section

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \quad \text{for class 1 or 2 cross sections} \qquad (4.13)$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,\min} f_y}{\gamma_{M0}} \quad \text{for class 3 cross sections} \qquad (4.14)$$

$$M_{c,Rd} = \frac{W_{eff,\min} f_y}{\gamma_{M0}} \quad \text{for class 4 cross-sections} \qquad (4.15)$$

where, $W_{el,min}$ and $W_{eff,min}$ corresponds to the fiber with the maximum elastic stress.

4. Shear

Resistance to vertical shear needs to satisfy the following condition:

$$V_{Ed} \leq V_{Rd} \qquad (4.16)$$

Shear resistance, V_{Rd}, is applied as $V_{b,Rd}$ when shear buckling is considered. Otherwise, it is applied as $V_{pl,Rd}$.

4.1 Plastic resistance to vertical shear

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} \qquad (4.17)$$

EN1993-1-1:2005 (6.18)

where,
γ_{M0} : the partial factor for resistance of cross-sections whatever the class is.
A_v : Refer to the table below
h_w : the depth of the web.
t_w : the web thickness.
η : the coefficient that includes the increase of shear resistance at web slenderness

[Table 4.6] Contribution from the web χw

Section	Av
Rolled I, H, Load parallel to web	$A-2bt_f+(t_w+2r)t_f \geq \eta h_w t$
Rolled channel sections, load parallel to web	$A-2bt_f+(t_w+r)t_f$
Rolled T-section, load parallel to web	$0.9(A-bt_f)$
Welded I, H and box sections, load parallel to web	$\eta\sum(h_w t_w)$
Welded I, H channel and box sections, load Parallel to flanges	$A-\sum(h_w t_w)$
Rolled rectangular hollow sections of uniform thickness: load parallel to depth load parallel to width	$Ah/(b+h)$ $Ab/(b+h)$
circular hollow sections and tubes of uniform thickness	$2A/\pi$

EN1993-1-1:2005 6.2.6(3)

[Table 4.7] Coefficient η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

4.2 Shear buckling resistance

Plates with $\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$ for an unstiffened web, or $\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$ for a stiffened web, should be checked for resistance to shear buckling and should be provided with transverse stiffeners at the supports. In midas Civil, longitudinal stiffener is not considered.

EN1993-1-5:2004 5.1(2)

- Limitation
  1. For Channel, H, B, and double Channel sections, shear buckling is verified only for internal parts.
  2. For Box section, shear buckling is provided for both major and minor direction.
  3. For Box and double channel section, if any of the part among webs and flanges satisfies the condition for shear buckling verification, shear buckling verification will be performed for entire parts.
  4. For box sections which have different flange thickness, shear buckling verification will be performed only for major axis.

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \quad (4.18)$$

EN1993-1-5:2004 (5.1)

(1) Contribution from the web $V_{bw,Rd}$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \quad (4.19)$$

EN1993-1-5:2004 (5.2)

where,

f_{yw} *: yield strength of the web.*

h_w *: clear web depth between flanges.*

t *: thickness of the plate.*

γ_{M1} *: partial factor for resistance of members to instability assessed by member checks.*

χ_w *: factor for the contribution of the web to the shear buckling resistance. In midas Civil, it is calculated by assuming the end support as non-rigid end post.*

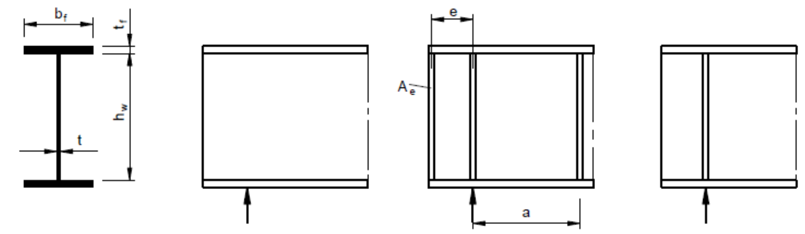
λw *: slenderness parameter.*

[Table 4.8] Contribution from the web χ_w

EN1993-1-5:2004 Table 5.1

Condition	Rigid end post	Non-rigid end post
$\overline{\lambda}_w < 0.83/\eta$	η	η
$0.83/\eta \le \overline{\lambda}_w < 1.08$	$0.83/\overline{\lambda}_w$	$0.83/\overline{\lambda}_w$
$\overline{\lambda}_w \ge 1.08$	$1.37/(0.7+\overline{\lambda}_w)$	$0.83/\overline{\lambda}_w$

EN1993-1-5:2004 Figure 5.1



Cross section notations a) No end post b) Rigid end post c) Non-rigid end post

[Fig.4.10] End supports

[Table 4.9] Calculation of λ_w

EN1993-1-1:2004 5.3(3)

Condition	$\overline{\lambda}_w$
Transverse stiffeners at supports only.	$\overline{\lambda}_w = \dfrac{h_w}{86.4t\varepsilon}$
Transverse stiffeners at supports and intermediate transverse or longitudinal stiffeners or both	$\overline{\lambda}_w = \dfrac{h_w}{37.4t\varepsilon\sqrt{k_\tau}}$

For webs with longitudinal stiffeners,

$$\overline{\lambda}_w \ge \frac{h_{wi}}{37.4t\varepsilon\sqrt{k_{\tau i}}} \quad (4.20)$$

EN1993-1-1:2004 5.3(5)

h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter λ_w of all subpanels within the web panel under consideration. ($k_{\tau st} = 0$)

$$\varepsilon = \sqrt{\frac{235}{f_y}} \quad (4.21)$$

k_τ : the minimum shear buckling coefficient for the web panel.

[Table 4.10] Calculation k_τ

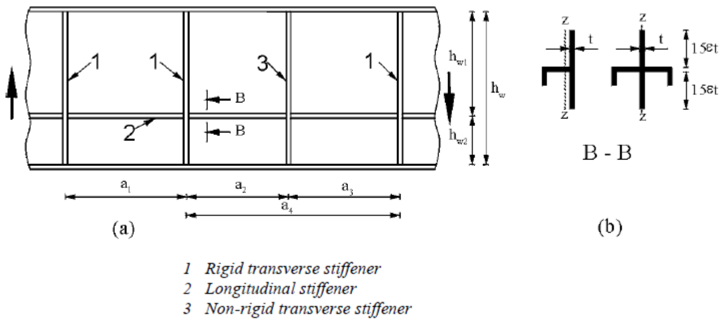
EN1993-1-1:2004 A.3

Longitudinal stiffeners num.	Condition	k_τ
= 0 or >2	$a/h_w \ge 1.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$a/h_w < 1.0$	$k_\tau = 4.00 + 5.34(h_w/a)^2 + k_{\tau st}$
1 or 2	$\alpha = a/h_w \ge 3.0$	$k_\tau = 5.34 + 4.00(h_w/a)^2 + k_{\tau st}$
	$\alpha = a/h_w < 3.0$	$k_\tau = 4.1 + \dfrac{6.3 + 0.18\dfrac{I_{sl}}{t^3 h_w}}{\alpha^2} + 2.2\sqrt[3]{\dfrac{I_{sl}}{t^3 h_w}}$

$$k_{\tau st}=9\left(\frac{h_w}{a}\right)^2\sqrt[4]{\left(\frac{I_{sl}}{t^3h_w}\right)^3}\geq\frac{2.1}{t}\sqrt[3]{\frac{I_{sl}}{h_w}} \tag{4.22}$$

a : the distance between transverse stiffeners
I_{sl} : the second moment of area of the longitudinal stiffener about z-axis.
When calculating k_τ , I_{sl} is replaced as 1/3 I_{sl}.
η : the coefficient that includes the increase of shear resistance at web slenderness

EN1993-1-1:2004 Figure 5.3



[Fig. 4.11] Web with transverse and longitudinal stiffeners

[Table 4.11] Calculation η

Steel Grade	η
S235 to S460	1.20
Over S460	1.00

(2) Contribution from the flange $V_{bf,Rd}$

$$V_{bf,Rd}=\frac{b_f t_f^2 f_{yf}}{c\gamma_{M1}}\left[1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right] \tag{4.23}$$

EN1993-1-1:2004 5.4(1)
EN1993-1-1:2004 (5.8)

where,
b_f and t_f are taken for the flange which provides the least axial resistance.
b_f being taken as not larger than $15\varepsilon_{tf}$ on each side of the web.
f_{yf} : yield strength of the flange.

$$c=a\left(0.25+\frac{1.6b_f t_f^2 f_{yf}}{th_w^2 f_{yw}}\right) \tag{4.24}$$

γ_{M1} : partial factor for resistance of members to instability assessed by member checks.
M_{Ed} : design bending moment.
M_f,R_d : the moment of resistance of the cross section consisting of the area of the effective flanges only.

[Table 4.12] Calculation of Mf,Rd

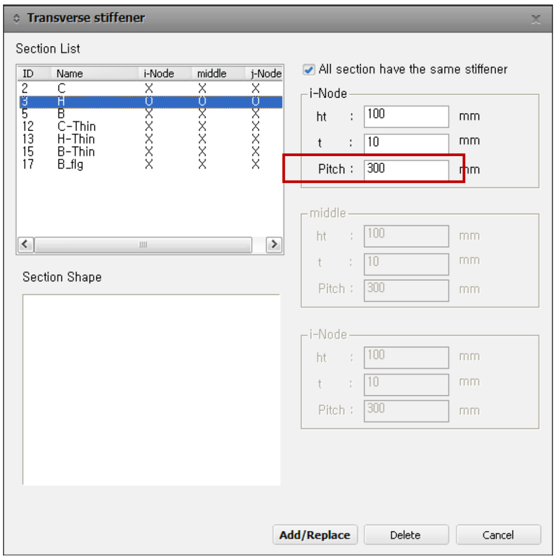
Condition	$M_{f,Rd}$
N_{Ed} = 0	$M_{f,Rd}$ is calculated as $M_{pl,Rd}$ but neglecting the web contribution.
N_{Ed} is present	$M_{f,Rd}$ is calculated by multiplying the reduction factor to the value of $M_{f,Rd}$ when N_{Ed}=0. $1-\frac{N_{Ed}}{\frac{(A_{f1}+A_{f2})f_{yf}}{\gamma_{M0}}}$

EN1993-1-1:2004 (5.9)

(3) Transverse stiffener

Transverse stiffener is specified by parts in the dialog box below.

▶ Design>Composite Steel Girder Design>Transverse Stiffener...



a = Space of rigid transverse stiffener

[Fig. 4.12] Transverse stiffener

4.3 Resistance to vertical shear

V_{Rd} is calculated in accordance with the value of h_w/t as written in the table below.

[Table 4.13] Calculation of V_{Rd}

	Condition	V_{Rd}
Unstiffened	$\frac{h_w}{t} \le \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$	$V_{Rd} = V_{b,Rd}$
Stiffened	$\frac{h_w}{t} \le \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{pl,Rd}$
	$\frac{h_w}{t} > \frac{31}{\eta}\varepsilon\sqrt{k_\tau}$	$V_{Rd} = V_{b,Rd}$

where,

$V_{pl,Rd}$ *: the plastic resistance to vertical shear.*

$V_{b,Rd}$ *: the shear buckling resistance.*

4.4 Interaction bending and vertical shear

(1) Consideration of interaction bending and vertical shear

EN1993-1-1:2004 7.1.(1)

If the following condition is satisfied, the interaction of bending and vertical shear will be verified.

$$\overline{\eta_3} = \frac{V_{Ed}}{V_{bw,Rd}} > 0.5 \qquad (4.25)$$

where,

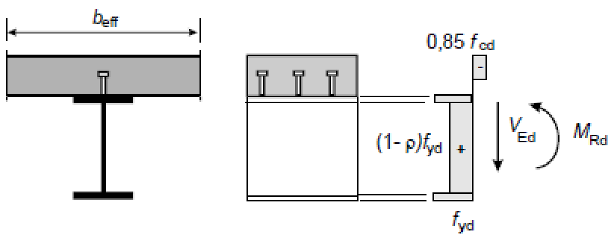
V_{Ed} *: the design shear force including shear from torque.*

$V_{bw,Rd}$ *: the design resistance for shear of contribution from the web.*

(2) For cross-sections in Class1 or 2

Apply the reduced design steel strength (1-ρ)f_{yd} in the shear area.

$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2 \tag{4.26}$$



[Fig. 4.13] Plastic stress distribution modified by the effect of vertical shear

(3) For cross-sections in Class3 and 4

- $\overline{\eta_3} \le 0.5$: M_{Rd}, N_{Rd} need not be reduced.
- $\overline{\eta_3} > 0.5$: The combined effects of bending and shear in the web of an I or box girder should satisfy.

$$\overline{\eta_1} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\overline{\eta_3} - 1\right)^2 \le 1.0 \tag{4.27}$$

EN1993-1-1:2004 (7.1)

$$\overline{\eta_1} = \frac{M_{Ed}}{M_{pl,Rd}} \ge \frac{M_{f,Rd}}{M_{pl,Rd}} \tag{4.28}$$

$$\overline{\eta_3} = \frac{V_{Ed}}{V_{bw,Rd}} \tag{4.29}$$

4.5 Check resistance to vertical shear

$$V_{Ed} \le V_{Rd} \tag{4.30}$$

where,

V_{Ed} : Design value of the shear force acting on the composite section.

V_{Rd} : Design value of the resistance of the composite section to vertical shear.

5. Bending and shear

EN1993-1-1:2005 6.2.8

(1) If $V_{Ed} < 0.5V_{pl,Rd}$, the effect of shear on moment resistance may be neglected.

(2) If $V_{Ed} \geq 0.5V_{pl,Rd}$, Yield strength should be reduced (1-ρ)fy for the shear area

where,

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd} - 1} \right)^2$$ *and* $V_{pl,Rd}$ *is calculated based on the equation (4.17).*

(3) if $V_{Ed} \geq 0.5V_{pl,Rd}$, for an I section with equal flange and bending about major axis

$$M_{y,v,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w} \right] f_y}{\gamma_{M0}} \quad \text{but,} \quad M_{y,V,Rd} \leq M_{y,c,Rd} \qquad (4.31)$$

$A_w = h_w t_w$

If A_w cannot be calculated, it is applied as A_{sz}(shear area) as per Eurocode3-1.

For equal flange H, T, B, P, SR, SB, 2L and 2C sections, verify the bending and shear. If the reduced moment resistance is calculated as negative value, the resistance will be applied as very small value and the results will be determined as NG. Torsion is not considered in midas Civil.

6. Bending and axial force

EN1993-1-1:2005 6.2.9

$M_{Ed} \leq M_{N,Rd}$ For Class 1 and Class 2 Cross sections (4.32)

$M_{N,Rd} = M_{pl,Rd}\left[1 - \left(N_{Ed} / N_{pl,Rd}\right)^2\right]$ For a rectangular solid section (4.33)

If the reduced moment resistance is calculated as negative value, the resistance will be applied as very small value and the results will be determined as NG. Doubly symmetrical I, H and other flanged sections, allowance for axial force need not be made if,

(1) Along Y-Y axis : must be satisfied both following two equations.

$$N_{Ed} \leq 0.25N_{pl,Rd} \qquad (4.34)$$

$$N_{Ed} \leq \frac{0.5h_w t_w f_y}{\gamma_{M0}} \qquad (4.35)$$

(2) Along Z-Z axis

$$N_{Ed} \leq \frac{h_w t_w f_y}{\gamma_{M0}} \qquad (4.36)$$

I and H sections (fastener holes are not considered)

(3) For cross-sections where bolt holes are not to be accounted for, the following approximations may be used for standard rolled I or H sections and for welded I or H sections with equal flanges:

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5a) \quad \text{but} \quad M_{N,y,Rd} \le M_{pl,y,Rd} \tag{4.37}$$

for n≤a:

$$M_{N,z,Rd} = M_{pl,z,Rd} \tag{4.38}$$

EN1993-1-1:2005 (6.36)

for n>a:

$$M_{N,z,Rd} = M_{pl,z,Rd}\left[1-\left(\frac{n-a}{1-a}\right)^2\right] \tag{4.39}$$

where,

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

$a = (A-2bt_f)/A$ *but a ≤0.5*

(4) For rectangular structural hollow sections of uniform thickness and for welded box sections with equal flanges and equal webs:

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5a_w) \quad \text{but} \quad M_{N,y,Rd} \le M_{pl,y,Rd} \tag{4.40}$$

$$M_{N,y,Rd} = M_{pl,y,Rd}(1-n)/(1-0.5a_f) \quad \text{but} \quad M_{N,z,Rd} \le M_{pl,z,Rd} \tag{4.41}$$

where,

$a_w = (A-2bt)/A$ *but aw ≤ 0.5 for hollow sections*

$a_w = (A-2bt_f)/A$ *but aw ≤ 0.5 for welded box sections*

$a_f = (A-2ht)/A$ *but af ≤ 0.5 for hollow sections*

$a_f = (A-2ht_w)/A$ *but af ≤ 0.5 for welded box sections*

(5) For bi-axial bending the following criterion may be used:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \le 1 \tag{4.42}$$

where

I and H sections :

$\alpha = 2$; $\beta = 5n$ *but* $\beta \ge 1$

EN1993-1-1:2005 (6.41)

circular hollow sections:

$\alpha = 2$; $\beta = 2$

rectangular hollow sections:

$\alpha = \beta = \dfrac{1.66}{1-1.13n^2}$ *; but* $\alpha = \beta \le 6$

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

(6) For Class 3 and Class 4 cross-sections

In the absence of shear force, for Class 3 cross-sections the maximum longitudinal stress shall satisfy the criterion

$$\sigma_{x,Ed} \le \frac{f_y}{\gamma_{M0}} \qquad (4.43)$$

EN1993-1-1:2005 6.2.9.2, 6.2.9.3

where, $\sigma_{x,Ed}$ *is the design value of the local longitudinal stress due to moment and axial force.*

For Class 4 sections, effective cross sections are applied for calculating stresses.

$$\frac{N_{Ed}}{A_{eff} f_y / \gamma_{MO}} + \frac{M_{y,Ed} + N_{Ed} e_{Ny}}{W_{eff,y,\min} f_y / \gamma_{MO}} + \frac{M_{z,Ed} + N_{Ed} e_{Nz}}{W_{eff,z,\min} f_y / \gamma_{MO}} \le 1 \qquad (4.44)$$

EN1993-1-1:2005 (6.44)

where,

A_{eff} *is the effective area of the cross-section when subjected to uniform compression.*

$W_{eff,min}$ *is the effective section modulus (corresponding to the fiber with the maximum elastic stress) of the cross-section when subjected only to moment about the relevant axis.*

e_N *is the shift of the relevant centroidal axis when the cross-section is subjected to compression only.*

EN1993-1-1:2005 6.2.10

7. Bending, shear and axial force

(1) If $V_{Ed} < 0.5V_{pl,Rd}$, The effect of shear on moment resistance may be neglected

(2) If $V_{Ed} \ge 0.5V_{pl,Rd}$, Yield strength should be reduced (1-ρ)fy for the shear area

where,

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd} - 1} \right)^2$$ *and* $V_{pl,Rd}$ *are calculated based on the equation (4.12).*

(3) If $V_{Ed} \ge 0.5V_{pl,Rd}$, for an I section with equal flange and bending about major axis

$$M_{y,v,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w} \right] f_y}{\gamma_{M0}} \quad \text{but} \quad M_{y,V,Rd} \le M_{y,c,Rd} \qquad (4.45)$$

If A_w cannot be calculated, it is applied as Asz(shear area) as per Eurocode3-1.

For equal flange H, T, B, P, SR, SB, 2L and 2C sections, verify the bending and shear.

If the reduced moment resistance is calculated as negative value, the resistance will be applied as very small value and the results will be determined as NG.

8. Buckling resistance of members

8.1 Uniform members in compression

(1) Buckling resistance

1) A compression member shall be verified against buckling as follows:

EN1993-1-1:2005
6.3.1.1

$$\frac{N_{Ed}}{N_{b,Rd}} \le 1 \qquad (4.46)$$

where N_{Ed} *is the design value of the compression force*

$N_{b,Rd}$ *is the design buckling resistance of the compression member.*

2) The design buckling resistance of a compression member

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$ for Class 1, and 3 cross-sections (4.47)

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}}$$ for Class 4 cross-sections (4.48)

where, χ *is the reduction factor for the relevant buckling mode*

(2) Buckling curve

1) For axial compression in members the value of χ for the appropriate non-dimensional slenderness λ should be determined from the relevant buckling curve according to:

EN1993-1-1:2005
6.3.1.2

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \le 1.0 \qquad (4.49)$$

where $\Phi = 0.5\left[1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right]$

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

$\overline{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}}$ *for Class 4 cross-sections*

α *is an imperfection factor*

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

[Table 4.14]: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor	0.13	0.21	0.34	0.49	0.76

EN1993-1-1:2005
Table 6.1

2) For slenderness $\bar{\lambda} \leq 0.2$ or for $\dfrac{N_{Ed}}{N_{cr}} \leq 0.04$ the buckling effects may be ignored and Only cross sectional check apply.

[Table 4.15] Selection of buckling curve

EN1993-1-1:2005
Table 6.2

Cross section		Limits (mm)	Buckling about axis	Buckling curve: S 235, S 275, S 355, S 420	Buckling curve: S 460
Rolled I sections	h/b>1,2	tf≤40	y-y	a	a_0
			z-z	b	a_0
		40 <tf ≤100	y-y	b	a
			z-z	c	a
	h/b≤1,2	tf ≤100	y-y	b	a
			z-z	c	a
		tf >100	y-y	d	c
			z-z	d	c
Welded I sections		tf ≤40	y-y	b	b
			z-z	c	c
		tf >40	y-y	c	c
			z-z	d	d
Hollow sections: circular, square, rectangular			any	a	a_0
Welded box sections		Generally (except as below)	any	b	b
		Thick welds: a>0,5 tf b/ tf<30 h/tf <30	any	c	c
Channel, T, Solid Rectangular, Solid circular			any	c	c
Angle section			any	b	b

(3) Slenderness for flexural buckling

1) The non-dimensional slenderness

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \quad \text{for Class 1, 2 and 3 cross-sections} \qquad (4.50)$$

EN1993-1-1:2005
6.3.1.3

$$\bar{\lambda} = \sqrt{\frac{A_{eff}\, f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{\dfrac{A_{eff}}{A}}}{\lambda_1} \quad \text{for Class 4 cross-sections} \qquad (4.51)$$

where, L_{cr} *is the buckling length in the buckling plane considered*

i *is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section*

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$ (f_y in N/mm²)

2) For flexural buckling the appropriate buckling curve should be determined from Table 4.15.

(4) Slenderness for torsional and torsional-flexural buckling

1) The non-dimensional slenderness

$$\bar{\lambda}_T = \sqrt{\frac{Af_y}{N_{cr}}} \text{ for Class 1, 2 and 3 cross-sections} \tag{4.52}$$

EN1993-1-1:2005 6.3.1.4

$$\bar{\lambda}_T = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \text{ for Class 4 cross-sections} \tag{4.53}$$

where $N_{cr} = N_{cr,TF}$ *but* $N_{cr} < N_{cr,T}$

$N_{cr,TF}$ *is the elastic torsional-flexural buckling force*

$N_{cr,T}$ *is the elastic torsional buckling force*

2) For torsional or torsional-flexural buckling the appropriate buckling curve may be determined from Table 4.15 considering the one related to the z-axis.

8.2 Uniform members in bending

(1) Buckling resistance

1) A laterally unrestrained beam subject to major axis bending shall be verified against lateral-torsional buckling as follows:

$$\frac{M_{Ed}}{M_{b,Rd}} \le 1.0 \tag{4.54}$$

EN1993-1-1:2005 6.3.2

where M_{Ed} *is the design value of the moment*

$M_{b,Rd}$ *is the design buckling resistance moment.*

2) The design buckling resistance of a compression member

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \tag{4.55}$$

where W_y *is the appropriate section modulus as follows:*

- $W_y = W_{pl,y}$ *for Class 1, 2 cross-sections*
- $W_y = W_{el,y}$ *for Class 3 cross-sections*
- $W_y = W_{eff,y}$ *for Class 4 cross-sections*

χ_{LT} *is the reduction factor for lateral-torsional buckling.*

(2) Lateral torsional buckling curves – General case

1) The non-dimensional slenderness

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^{\ 2} - \bar{\lambda}_{LT}^{\ 2}}} \le 1.0 \tag{4.56}$$

EN1993-1-1:2005 6.3.2.2

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

α_{LT} *is an imperfection factor*

$\bar{\lambda}_{LT} = \sqrt{\dfrac{W_y f_y}{M_{cr}}}$

M_{cr} *is the elastic critical moment for lateral-torsional buckling*

2) M_{cr} is based on gross cross sectional properties and takes into account the loading conditions, the real moment distribution and the lateral restraints.

[Table 4.16] Imperfection factors for lateral torsional buckling curves

Buckling curve	a	b	c	d
Imperfection factor	0.21	0.34	0.49	0.76

EN1993-1-1:2005 Table 6.3

[Table 4.17] Selection of buckling curves for cross sections

Cross section	limits	Buckling curve
Rolled I sections	h/b≤2	a
	h/b>2	b
Welded I sections	h/b≤2	c
	h/b>2	d
Other cross sections		d

EN1993-1-1:2005 Table 6.4

3) For slenderness $\bar{\lambda}_{LT} \le 0.2$ or $M_{Ed}/M_{cr} \le 0.04$ lateral torsional buckling effects may be ignored and only cross sectional checks apply.

8.3 Uniform members in bending and axial compression

EN1993-1-1:2005 6.3.3

(1) Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \le 1.0 \tag{4.57}$$

EN1993-1-1:2005 (6.61)

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} \le 1.0 \tag{4.58}$$

where

N_{Ed}, $M_{y,Ed}$ *and* $M_{z,Ed}$ *are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively*

$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ *are the moments due to the shift of the centroidal axis according to 6.2.9.3 for class 4 sections, see Table 4.18.*

χ_y , χ_z *are the reduction factors due to flexural buckling from 6.3.1*

χ_{LT} *is the reduction factors due to lateral torsional buckling from 6.3.2*

k_{yy} , k_{yz} , k_{zy} , k_{zz} *are the interaction factors*

※ If flexural buckling check is determined as NG, the verification above will not be applied.

[Table 4.18] Values for $N_{Rk} = f_y A_i, M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

EN1993-1-1:2005 Table 6.7

Class	1	2	3	4
A_i	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	$e_{N,y} N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z} N_{Ed}$

$k_{yy}, k_{yz}, k_{zy}, k_{zz}$ depend on one method between Annex A and B.

[Table 4.19] Interaction factors k_{ij}

EN1993-1-1:2005 Annex A, Table A.1

Interaction factors	Design assumptions	
	elastic cross-sectional properties Class 3, class 4	plastic cross-sectional properties Class 1, class 2
k_{yy}	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{yz}} 0,6 \sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{mz} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{zy}} 0,6 \sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{zz}}$

where,

$$\mu_y = \frac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-x_y \dfrac{N_{Ed}}{N_{cr,y}}}, \mu_z = \frac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-x_z \dfrac{N_{Ed}}{N_{cr,z}}}$$

$$w_y = \frac{W_{pl,y}}{W_{el,y}} \le 1,5, w_z = \frac{W_{pl,z}}{W_{el,z}} \le 1,5$$

$$n_{pl} = \frac{N_{Ed}}{N_{Rk} / \gamma_{M1}},$$

C_{my} *see Table A.2.*

$$a_{LT} = 1 - \frac{I_T}{I_y} \geq 0$$

$$C_{yy} = 1 + (w_y - 1)\left[\left(2 - \frac{1,6}{w_y} C_{my}^2 \bar{\lambda}_{\max} - \frac{1,6}{w_y} C_{my}^2 \bar{\lambda}_{\max}^2\right) n_{pl} - b_{LT}\right] \geq \frac{W_{el,y}}{W_{pl,y}}$$

with $b_{LT} = 0,5 a_{LT} \bar{\lambda}_0^2 \frac{M_{y,Ed}}{x_{LT} M_{pl,y,Rd}} \frac{M_{z,Ed}}{M_{pl,z,Rd}}$

$$C_{yz} = 1 + (w_z - 1)\left[\left(2 - 14 \frac{C_{mz}^2 \bar{\lambda}_{\max}^2}{w_z^5}\right) n_{pl} - c_{LT}\right] \geq 0,6 \sqrt{\frac{w_z}{w_y}} \frac{W_{el,z}}{W_{pl,z}}$$

with $c_{LT} = 10 a_{LT} \frac{\bar{\lambda}_0^2}{5 + \bar{\lambda}_z^4} \frac{M_{y,Ed}}{C_{my} x_{LT} M_{pl,y,Rd}}$

$$C_{zy} = 1 + (w_y - 1)\left[\left(2 - 14 \frac{C_{my}^2 \bar{\lambda}_{\max}^2}{w_y^5}\right) n_{pl} - d_{LT}\right] \geq 0,6 \sqrt{\frac{w_y}{w_z}} \frac{W_{el,y}}{W_{pl,y}}$$

with $d_{LT} = 2 a_{LT} \frac{\bar{\lambda}_0}{0,1 + \bar{\lambda}_z^4} \frac{M_{y,Ed}}{C_{my} x_{LT} M_{pl,y,Rd}} \frac{M_{z,Ed}}{C_{mz} M_{pl,z,Rd}}$

$$C_{zz} = 1 + (w_z - 1)\left[\left(2 - \frac{1,6}{w_z} C_{mz}^2 \bar{\lambda}_{\max} - \frac{1,6}{w_z} C_{mz}^2 \bar{\lambda}_{\max}^2\right) n_{pl} - e_{LT}\right] \geq \frac{W_{el,z}}{W_{pl,z}}$$

with $e_{LT} = 1,7 a_{LT} \frac{\bar{\lambda}_0}{0,1 + \bar{\lambda}_z^4} \frac{M_{y,Ed}}{C_{my} x_{LT} M_{pl,y,Rd}}$

$$\bar{\lambda}_{\max} = \max \begin{cases} \bar{\lambda}_y \\ \bar{\lambda}_z \end{cases}$$

$\bar{\lambda}_0$ = *non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment. i.e.* $\Psi_y = 1,0$ *in Table A.2 → In Gen, calculated like* $\bar{\lambda}_{LT}$.

$\bar{\lambda}_{LT}$ = *non-dimensional slenderness for lateral-torsional buckling*

If $\bar{\lambda}_0 \leq 0,2\sqrt{C_1} \sqrt[4]{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \frac{N_{Ed}}{N_{cr,TF}}\right)}$, $C_{my} = C_{my,0}$ $C_{mz} = C_{mz,0}$ $C_{mLT} = 1.0$

If $\bar{\lambda}_0 > 0,2\sqrt{C_1} \sqrt[4]{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \frac{N_{Ed}}{N_{cr,TF}}\right)}$, $C_{my} = C_{my,0} + (1 - C_{my,0}) \frac{\sqrt{\varepsilon_y} a_{LT}}{1 + \sqrt{\varepsilon_y} a_{LT}}$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = C_{my}^2 \frac{a_{LT}}{\sqrt{\left(1 - \frac{N_{Ed}}{N_{cr,z}}\right)\left(1 - \frac{N_{Ed}}{N_{cr,T}}\right)}} \geq 1$$

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

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}} \frac{A_{eff}}{W_{eff,y}}$ *for class 4 cross-sections*

$N_{cr,y}$ = *elastic flexural buckling force about the y-y axis*

$N_{cr,z}$ = *elastic flexural buckling force about the z-z axis*

$N_{cr,T}$ = *elastic torsional buckling force*

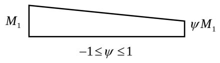
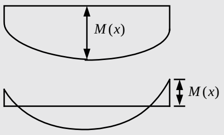
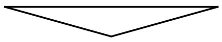
I_T = *St. Venant torsional constant*

I_y = *second moment of area y-y axis*

C_1 *; see EN 1993-1-1:1992 Table F.1.1 and F.1.2*

EN1993-1-1:2005
Annex A, Table A.2

[Table 4.20] Equivalent uniform moment factors $C_{mi,0}$

Moment Diagram	$C_{mi,0}$
	$C_{mi,0} = 0,79 + 0,21\psi_i + 0,36(\psi_i - 0,33)\dfrac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + \left(\dfrac{\pi^2 EI_i \lvert\delta_x\rvert}{L^2 \lvert M_{i,Ed}(x)\rvert} - 1\right)\dfrac{N_{Ed}}{N_{cr,i}}$ $M_{i,Ed}(x)$ is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$ $\lvert\delta_x\rvert$ is the maximum member displacement along the member.
	$C_{mi,0} = 1 - 0,18\dfrac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + 0,03\dfrac{N_{Ed}}{N_{cr,i}}$

If All Moments are zero, assumed that $\psi_i = 1.0$

$$N_{crT} = \frac{1}{i_s^2}\left(\frac{\pi^2 EI_z a^2}{L_t^2} + \frac{\pi^2 EI_w}{L_t^2} + GI_t\right) \tag{4.59}$$

where:

$i_s^2 = i_y^2 + i_z^2 + a^2$

$a^2 = y_0^2 + z_0^2$

I_w *is warping constant.*

I_t *is St. Venant torsional constant.*

y_0, z_0 *are the coordinates of the shear centre with respect to the centroid (see Figure 2.1). For a doubly symmetric cross-section, the shear centre coincides with the centroid; then yo = 0 and zo = 0*

$$G = \frac{E}{2(1+\nu)}$$

ν *is Poisson's ratio.*

L_t ; $Max[L_y, L_z]$ *for Column.*

L_y *for beam.*

L_y, L_z *is unbraced lengths.*

Serviceability Limit States

1. Deflection

Steel structures and components shall be so proportioned that deflections are within the limits agreed between the client, the designer and the competent authority as being appropriate to the intended use and occupancy of the building and the nature of the materials to be supported.

1.1 Limiting values

$$\delta_{max} = \delta_1 + \delta_2 - \delta_0 \tag{4.60}$$

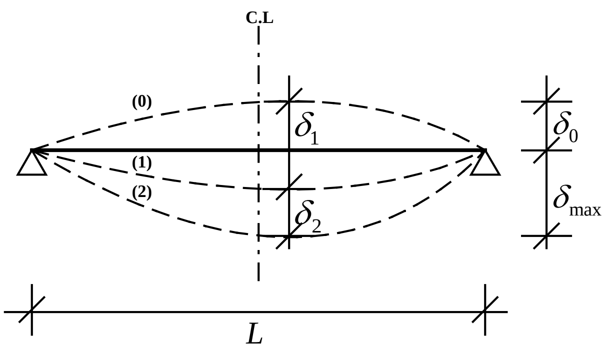
where,

δ_{max} *is the sagging in the final state relative to the straight line joining the supports.*

δ_0 *is the pre-camber (hogging) of the beam in the unloaded state, (state0).*

δ_1 *is the variation of the deflection of the beam due to the permanent loads immediately after loading, (state1).*

δ_2 *is the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load, (state2).*



[Fig. 4.14] Vertical deflections to be considered

[Table 4.21] Recommended limiting values for vertical deflections

Conditions	Limits δ_{max}	Limits δ_2
-Roofs generally	L/200	L/250
-Roofs frequently carrying personnel other than for maintenance	L/250	L/300
-Floors generally	L/250	L/300
-Floors and roofs supporting plaster or other brittle finish or non-flexible partitions	L/250	L/350
-Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	L/400	L/500
-where δ_{max} can impair the appearance of the building	L/250	-

where, L is the span of the beam.

➔ In midas Civil, only δ_{max} is verified.

-For buildings the recommended limits for horizontal deflections at the tops of the columns are as follows.

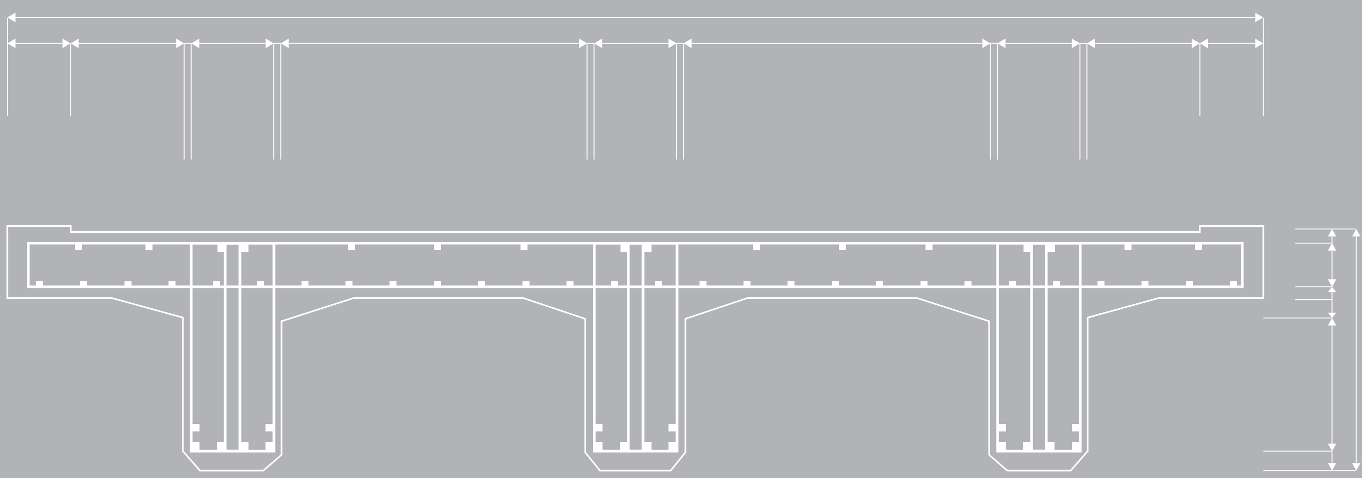
[Table 4.22] Recommended limiting values for horizontal deflections

Conditions	Limits
-Portal frames without gantry cranes	h/150
-Other single storey buildings	h/300
-In a multistory building	
-In each storey	h/300
-On the structure as a whole	h0/500
where h is the height of the column or of the storey And h0 is the overall height of the structure.	

Chapter 5.

Reinforced Concrete Frame Design

EN 1992-2



Chapter 5.

RC Frame Design (EN 1992-2)

RC girder and column need to be designed to satisfy the following limit states.

Ultimate Limit States

- Moment Resistance
- Shear Resistance

Serviceability Limit States

- Stress of Cross Section
- Crack Width
- Deflection

Ultimate Limit States

1. Moment resistance

Limit state of moment resistance should satisfy the condition, $M_{Ed} \le M_{Rd.}$

Moment resistance, M_{Rd}, is calculated using the strain compatibility method as shown below.

1.1 Design strength of material

(1) Design compressive strength of concrete

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \qquad (5.1)$$

EN1992-1-1:2004 3.1.6(1)

where,

α_{cc}: *The coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied.*

f_{ck} : *The characteristic compressive cylinder strength of concrete at 28 days.*

γ_c : *The partial safety factor for concrete.*

(2) Design yield strength of reinforcement

$$f_{yd} = f_{yk} / \gamma_s \qquad (5.2)$$

EN1992-1-1:2004 3.2.7(2)

where,

f_{yk}: *The characteristic yield strength of reinforcement.*

γ_s : *The partial safety factor for reinforcement or prestressing steel.*

• Partial factors for materials γ_c, γ_s / Coefficient for long term α_{cc}, α_{ct}

Default values of partial factors for materials are shown in the table below. The values can be entered by the user.

[Table 5.1] Partial factors for materials for ULS

Design Situations	γ_c for concrete	γ_s for reinforcing steel
Persistent & Transient	1.5	1.15
Accidental	1.2	1.0

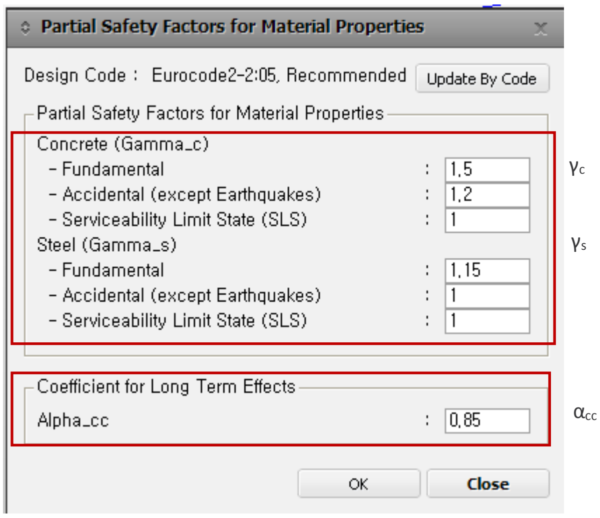
EN1992-1-1:2004 Table 2.1N

Partial safety factor γ_c, γ_s / Coefficient for long term α_{cc}

Main design parameters for materials can be entered in Partial Safety Factor for Material properties dialog box. Among the input values, α_{cc} is considered when calculating moment resistance in Ultimate Limit State and it is applied as 1.0 for shear and torsional resistance.

The coefficient for long term, α_{cc}, is considered during calculating moment resistance in Ultimate Limit State design. It is applied as 1.0 in the calculation of shear resistance.

◖ *Design > RC Design > Partial Safety Factor for Material properties ...*



[Fig. 5.1] Modify Design Parameters Input Dialog

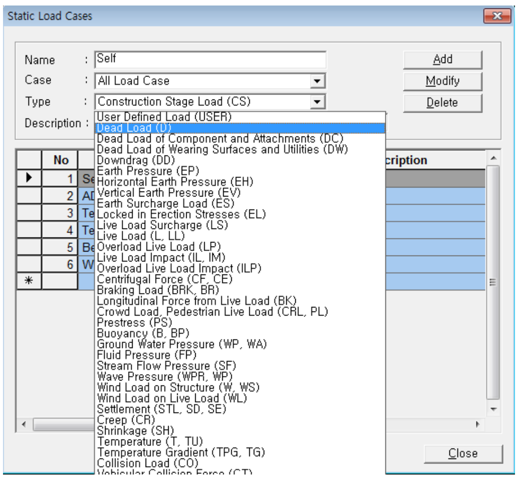
Partial factors for materials for 'persistent & transient' and 'Accidental' design situations are given in the table 5.2.

[Table 5.2] Classification of design situations

Design situations	Description
Persistent & Transient	Load combination not "Accidental situation"
Accidental	Load combination include following load case type, Live Load Impact (IL, IM) Collision Load (CO) Vehicular Collision Force (CT) Vessel Collision Force (CV)

Load case type need to be specified in Static Load Cases dialog box.

◖ *Load>Static Load Cases...*

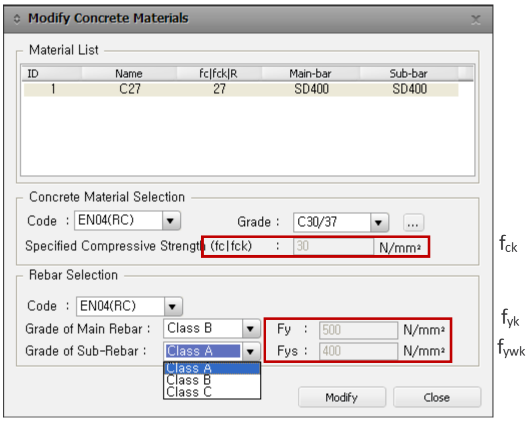


[Fig. 5.2] Static Load Cases Input Dialog

□ Strength of Concrete/Reinforcement

Define the material strengths of concrete and steel in Modify Concrete Material dialog box.

Design > RC Design > Modify Concrete Material



[Fig. 5.3] Define f_{ck}, f_{yk}, f_{ywk}

Select 'None' in the Code field and enter the name of the material to be used in the Name field. Then, each data field is activated and the strength of materials can be entered.

In midas Civil, characteristic strength (f_{ck}) in concrete is limited by national annex as shown below. If the strength of the material exceeds the permitted range, the corresponding members are excluded in concrete code design.

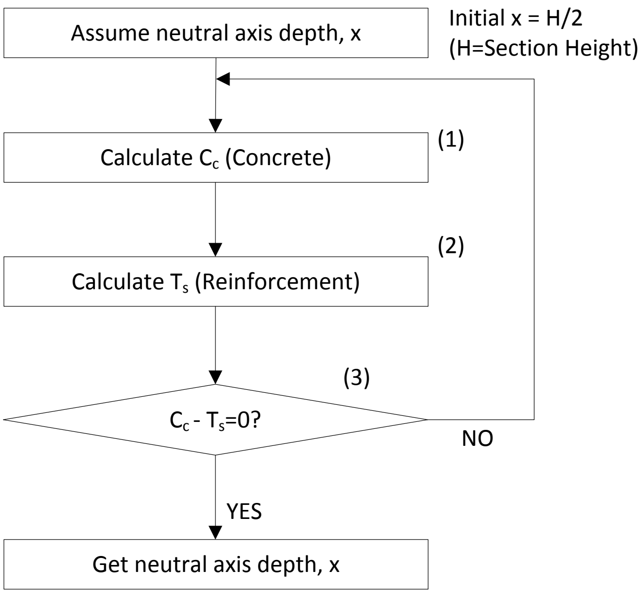
[Table 5.3] Limit strength of fck (MPa)

National Annex	Min	Max
Recommended	30.0	70.0
UK	25.0	70.0
Italy	25.0	60.0

1.2 Calculate neutral axis depth

Calculate the position of neutral axis by iterative approach as shown in the figure below.

In midas Civil, singly reinforced beam design method is applied in the calculation of neutral axis and flexural strength for conservative design.



[Fig. 5.4] Flow chart to calculate neutral axis depth, x

(1) Calculate force of concrete, C_c.

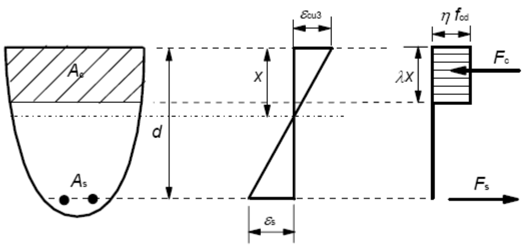
$$C_c = \eta f_{cd} \int_{dA} \lambda x \tag{5.3}$$

where,
λ: The effective height of the compression zone factor.
η: The effective strength factor.
x: The neutral axis depth.

[Table 5.4] Effective height and strength factor by compressive strength

Condition	λ	η
$f_{ck} \le$ 50MPa	0.8	1.0
50 < $f_{ck} \le$ 90MPa	0.8-(f_{ck}-50)/400	1.0-(f_{ck}-50)/200
f_{ck} > 90MPa	0.7	0.8

• In midas Civil, a rectangular stress distribution is used as shown in the figure below. (Ultimate strain of concrete ε_{cu}= ε_{cu1})



[Fig. 5.5] Rectangular stress distribution

(2) Calculate force of reinforcement, T_s.

$$T_s = A_s f_s \tag{5.4}$$

where,
A_s,: The cross sectional area of tensile reinforcement.
f_s ,: The stress of tensile and compressive reinforcement.

In order to calculate the stress of reinforcing steel, f_s , calculate the appropriate strain by the strain compatibility condition. And then calculate the corresponding stresses in the stress-strain diagram.
Calculation method of strain and stress is as follow.

• Calculate the strains of reinforcement by assuming a linear strain distribution and the strain of ε_{cu3} at the extreme fiber of the concrete in compression.

$$\varepsilon_s = \frac{d_t - x}{x} \varepsilon_{cu} \tag{5.5}$$

where,
ε_s : The strain of tensile reinforcement.
ε_{cu}: The ultimate compressive strain in the concrete. ($\varepsilon_{cu} = \varepsilon_{cu1}$)

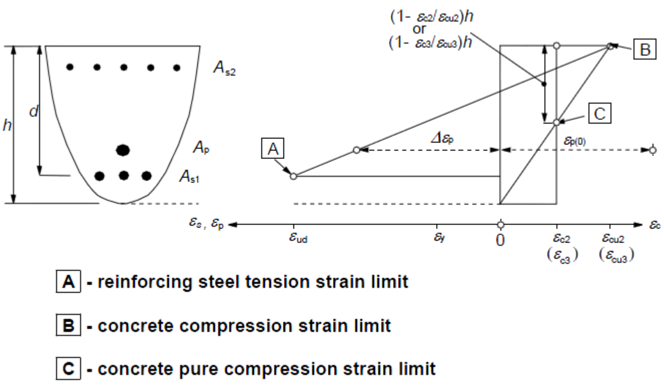
EN1992-1-1:2004
Table 3.1

[Table 5.5] Effective height and strength factor by compressive strength

Condition	ε_{cu1}
$f_{ck} \le 50MPa$	0.0035
$50 < f_{ck} \le 90MPa$	$[2.8+27\{(98-f_{cm})/100\}^4]/1000$, $f_{cm}=f_{ck}+8MPa$
$f_{ck} > 90MPa$	0.0028

x : The neutral axis depth.
d_t : Distance from the tensile rebar to the extreme top fiber of the element
d_c : Distance from the compressive rebar to the extreme top fiber of the element

EN1992-1-1:2004
Figure 6.1



[Fig. 5.6] Possible strain distributions in the ultimate limit state

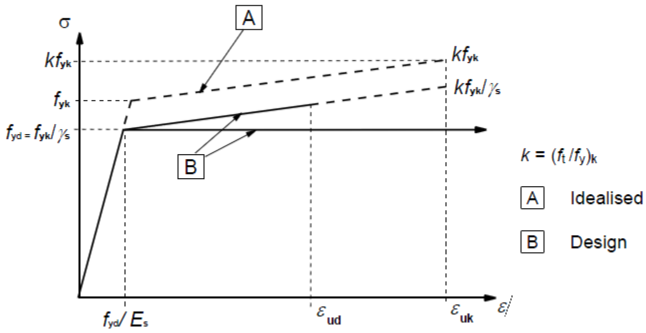
• Calculate the reinforcement stresses appropriate to the calculated reinforcement strains. (from the stress-strain idealizations)

$$f_s = \begin{cases} \varepsilon_s E_s & (\varepsilon_s \le \varepsilon_{yd}) \\ f_{yd} & (\varepsilon_s > \varepsilon_{yd}) \end{cases} \tag{5.6}$$

$$\varepsilon_{yd} = f_{yd} / E_s \tag{5.7}$$

$$f_{yd} = f_{yk} / \gamma_s \quad (5.8)$$

where,
E_s : The design value of modulus of elasticity of reinforcement.
f_{yd} : The design yield strength of reinforcement. (See 1.1(2))
ε_{yd} : The yield strain of reinforcement.



[Fig. 5.7] Idealized and design stress-strain diagram for reinforcing steel

EN1992-1-1:2004
Figure 3.8

(4) Check if resultant force is zero.

Determine the neutral axis position by iterative approach of the clause (1) and (2) until the compressive strength ($C=C_c$) and tensile strength ($T=T_s$) become identical.

In midas Civil, convergence condition for "C = T" is applied as follows.

• Convergence condition :

Error! Bookmark not defined. $\left|\frac{C}{T} - 1.0\right| < 0.01 \quad (Tolerance)$ (5.9)

$$C = C_c, \quad T = T_s \quad (5.10)$$

• Reassume neutral axis depth by "Bisection method (Numerical analysis)" before meet following stop condition.

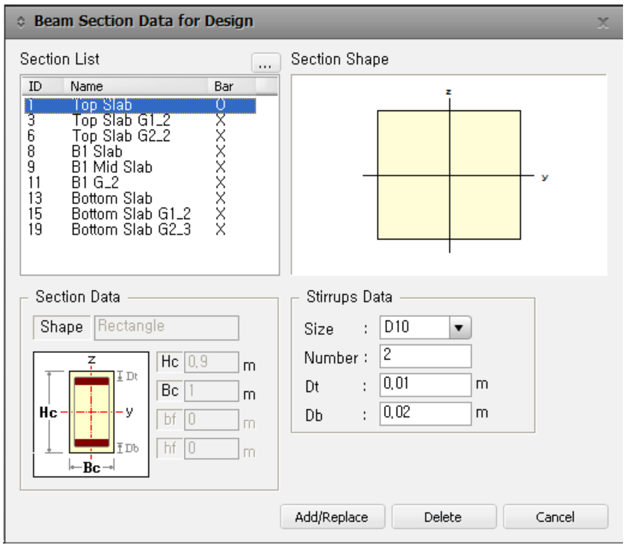
[Table 5.6] Stop condition for iterative approach

Stop condition	Description
Converge	$\frac{C}{T} - 1.0 < 0.01$
Not converge	Repeat count > 20 → Output "Not converge" in Message window. → Need to modify model as following. - Increase section size. - Modify the rebar information (position, numbers, spacing, etc.)

Beam Section Data for Design

Define the section and stirrup data to be applied in concrete code design. In midas Civil, rebar ratio is determined in between minimum and maximum rebar ratio.

Design > RC Design > Beam Section for Design



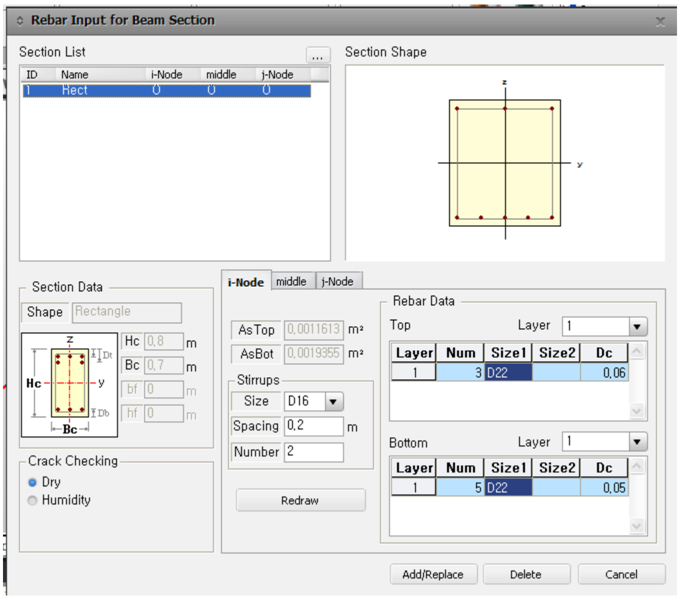
[Fig. 5.8] Beam Section for Design Dialog

Where, Dt and Db represent the distance from the rebar center to top and bottom fiber respectively.

Rebar Input for Beam Section for checking

Define rebar data for concrete code checking. In midas Civil, both top and bottom rebar must be defined to perform concrete code checking.

Design > RC Design > Beam Section Data for Checking



[Fig. 5.9] Beam Section for Design Dialog

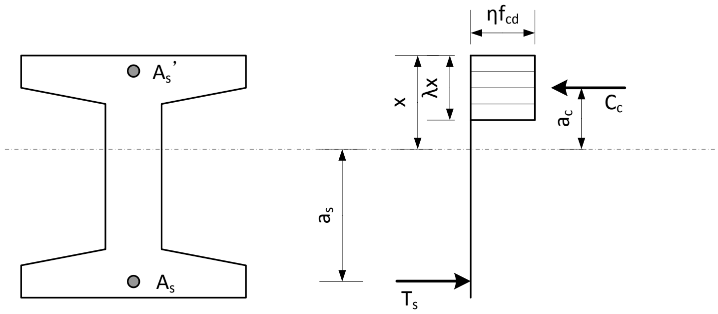
1.3 Calculate moment resistance M_{Rd}

Once the neutral axis is calculated, moment resistance can be calculated by multiplying the axial forces and eccentricity from the neutral axis.

$$M_{Rd} = C_c a_c + T_s a_s \tag{5.11}$$

where,

a_c, a_s, : The distance from neutral axis depth(or centroid), x to concrete, reinforcement rebar.



[Fig. 5.10] Forces and distances from neutral axis depth for M_{Rd}

In midas Civil, singly reinforced beam design method is applied for conservative design.

Flexural moment is calculated from C_c and T_s which generate the same amount of moment about neutral axis. Theoretically the flexural moment will be identical at any position of the cross section. In midas Civil, flexural moment is calculated at the centroid of the cross section.

1.4 Check moment resistance

$$M_{Ed} \leq M_{Rd} \tag{5.12}$$

where,

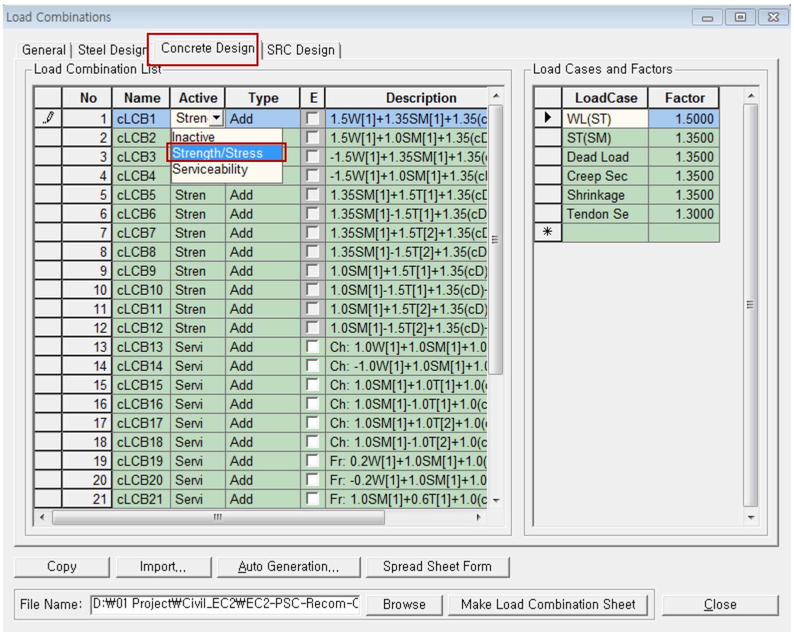
M_{Ed} : Design value of the applied internal bending moment.

M_{Rd} : Design moment resistance.

• Design load combination

Load combinations used in concrete code design are generated from Results>Load combinations>Concrete Design tab. Load combinations specified as “Strength/Stress” in Active column are applied for ultimate limit state design.

Results>Load Combinations...



[Fig. 5.11] Load Combinations Input Dialog

1.5 Verification of rebar ratio

(1) Minimum rebar ratio

In midas Civil, minimum rebar ratio of longitudinal reinforcement is applied as shown below.

$$A_{s,\min} = Min\left(0.26\frac{f_{ctm}}{f_{yk}}b_t d,\ 0.0013b_t d\right) \qquad (5.13)$$

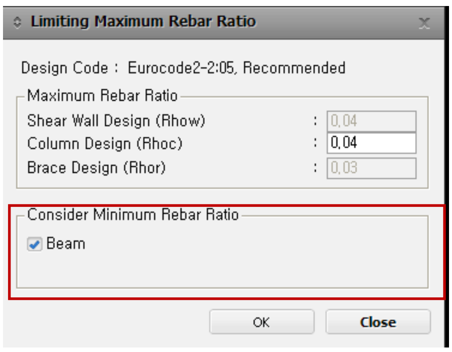
EN1992-1-1:2004 9.2.1.1(1)

where,

b_t : The mean width of the tension zone. For T-shape beam when top flange is in compression, b_t is applied as web width.

The verification of minimum rebar ratio can be selectively performed based on the option in Limiting Maximum Rebar Ratio dialog box.

Design>RC Design> Limiting Rebar Ratio ...



[Fig. 5.12] Limiting Maximum Rebar Ratio Dialog

(2) Maximum rebar ratio

In midas Civil, maximum rebar ratio is applied as below.

EN1992-1-1:2004 9.2.1.1(3)

$$A_{s,\max} = 0.04A_c \qquad (5.14)$$

2. Shear resistance

Limit state of shear resistance should satisfy the condition, $V_{Ed} \le V_{Rd}$.

Shear resistance, V_{Rd}, is calculated as follows.

2.1 Design strength of material

(1) Design compressive strength of concrete.

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \tag{5.15}$$

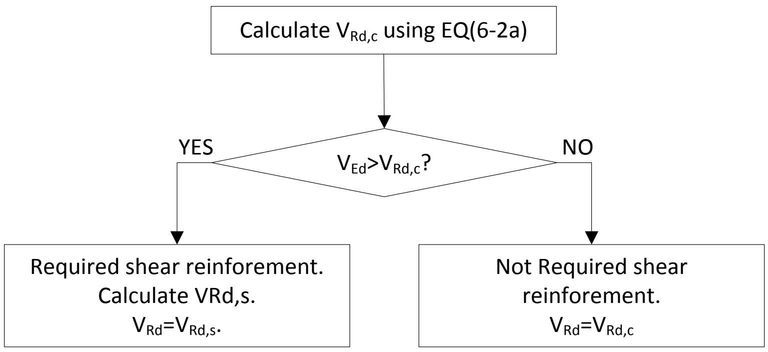
Using α_{cc}=1.0 for shear regardless of input value.

(2) Design yield strength of reinforcement.

$$f_{yd} = f_{yk} / \gamma_s \tag{5.16}$$

Refer to the clause 1.1 for detail explanation of material strength.

2.2 Calculate shear resistance V_{Rd}



[Fig. 5.13] Flowchart to calculate V_{Rd}

When design shear reinforcement is not required in the verification of shear, shear resistance is calculated by concrete only. If design shear force exceeds shear resistance calculated from concrete, the shear resistance is calculated by shear reinforcement only.

(1) Calculate $V_{Rd,c}$

$$V_{Rd,c} = \left[C_{Rd,c} k (100 \rho_l f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp} \right] b_w d \tag{5.17}$$

EN1992-1-1:2004
6.2.2(1)
(6.3N)

$$V_{Rd,c} \ge (v_{\min} + k_1 \sigma_{cp}) b_w d \tag{5.18}$$

where,

$V_{Rd,c}$ *: The design shear resistance without shear reinforcement.*

b_w *: The smallest width of the cross-section in the tensile area.*

d *: The effective depth of cross-section.*

d_s *: Distance from the centroid of tensile rebar to the extreme fiber of cross-section*

h *: Height of section.*

σ_{cp} *:* N_{Ed} / A_c, *In beam design,* σ_{cp} *is applied as zero since axial force is not considered.*

$$C_{Rd,c} = \frac{0.18}{\gamma_c} \quad (5.19)$$

EN1992-1-1:2004 (6.8), (6.13) (6.9), (6.14) (6.12), (6.15)

$$k = 1 + \sqrt{200/d} \leq 2.0 \quad (5.20)$$

$$\rho_l = \frac{A_{sl}}{b_w d} \leq 0.02 \quad (5.21)$$

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{\ 1/2} \quad (5.22)$$

$$f_{ctd} = \frac{\alpha_{ct} f_{ck}}{\gamma_c} \quad (5.23)$$

(2) Calculate $V_{Rd,s}$

Shear resistance of members with shear reinforcement can be calculated depending on the type of shear reinforcement.

[Table 5.7] $V_{Rd,s}$ and $V_{Rd,max}$, $A_{sw,max}$

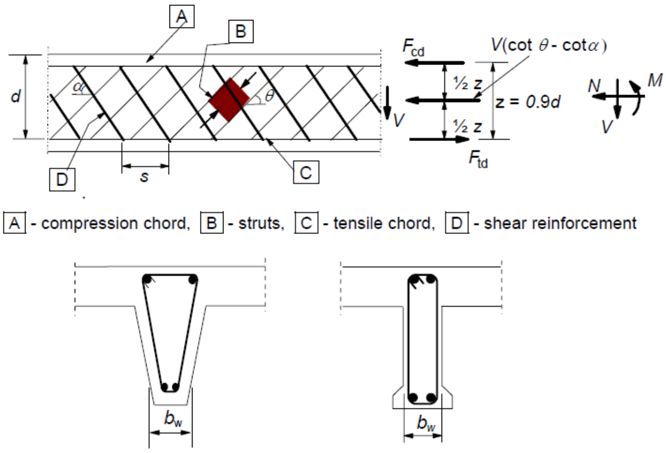
Type	Vertical shear reinforcement
$V_{Rd,s}$	$\frac{A_{sw}}{s} z f_{ywd} \cot\theta$
$V_{Rd,max}$	$\frac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot\theta + \tan\theta}$
$A_{sw,max}$	$\frac{A_{sw,\max} f_{ywd}}{b_w s} \leq \frac{1}{2} \alpha_{cw} v_1 f_{cd}$

where,

$V_{Rd,s}$: The design value the shear force which can be sustained by the yielding shear reinforcement.

Θ : The angle between the concrete compression strut and the beam axis perpendicular to the shear force.

α : The angle between shear reinforcement and the beam axis perpendicular to the shear force. In midas Civil, α is always applied as 90 degree.



EN1992-1-1:2004 Figure 6.5

[Fig. 5.14] Truss model and notation for shear reinforced members

A_{sw} : The cross-sectional area of the shear reinforcement.

s : The spacing of stirrups.

z : Inner lever arm, $z=0.9d$.

f_{ywd} : The design yield strength of the shear reinforcement.

v_1 : Strength reduction factor for concrete cracked in shear.

[Table 5.8] Strength reduction factor for concrete cracked in shear, ν_1

National Annex	$f_{ywd} \geq 0.8_{fywk}$	$f_{ywd} < 0.8_{fywk}$	
		$f_{ck} < 60MPa$	$f_{ck} \geq 60MPa$
Recommended	$0.6\left(1-\dfrac{f_{ck}}{250}\right)$	0.6	$0.9-\dfrac{f_{ck}}{200} > 0.5$
British	$0.6\left(1-\dfrac{f_{ck}}{250}\right)$	$0.54(1-0.5\cos\alpha)$	$\left(0.84-\dfrac{f_{ck}}{200}\right)(1-0.5\cos\alpha) > 0.5$

EN1992-1-1:2004 (6.10.aN),(6.10.bN)

[Table 5.9] Strength reduction factor for concrete cracked in shear, ν_1

National Annex	$f_{ywd} \geq 0.8_{fywk}$		$f_{ywd} < 0.8_{fywk}$	
	$f_{ck} \leq 70MPa$	$f_{ck} > 70MPa$	$f_{ck} < 60MPa$	$f_{ck} \geq 60MPa$
Italy	0.5	$0.7\left(1-\dfrac{f_{ck}}{250}\right)$	0.7	$\dfrac{0.9-\dfrac{f_{ck}}{200}}{0.85} > 0.5$

α_{cw} : Coefficient taking account of the state of the stress in the compression chord. α_{cw} is always applied as 1.0 in beam design.

[Table 5.10] Coefficient α_{cw}

Condition	α_{cw}
$0<\sigma_{cp} \leq 0.25f_{cd}$	$1+\sigma_{cp}/f_{cd}$
$0.25\ f_{cd} <\sigma_{cp} \leq 0.5f_{cd}$	1.25
$0.5\ f_{cd} <\sigma_{cp} \leq 1.0f_{cd}$	$2.5(1-\sigma_{cp}/f_{cd})$

EN1992-1-1:2004 (6.11.aN)~(6.11.cN)

σ_{cp} : The mean compressive stress, measured positive, in the concrete due to the design axial force. In beam design, σ_{cp} is applied as zero since axial force is not considered.

(3) Calculate shear resistance V_{Rd}.

• The shear resistance of a member with shear reinforcement.

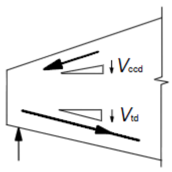
$$V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td} \tag{5.24}$$

EN1992-1-1:2004 (6.1)

where,

V_{ccd} : The design value of the shear component of the force in the compression area, in the case of an inclined compression chord.

V_{td} : The design value of the shear component of the force in the tensile reinforcement, in the case of an inclined tensile chord.



[Fig. 5.15] Shear component for members with inclined chords

EN1992-1-1:2004 Figure 6.2

In midas civil, inclined chord is not considered. Therefore the shear resistance is calculated using shear reinforcement only.

$$V_{Rd} = V_{Rd,s} \tag{5.25}$$

EN1992-1-1:2004 6.2.1(5)

• In regions of the member where $V_{Ed} \leq V_{Rd,c}$ no calculate shear reinforcement is necessary.

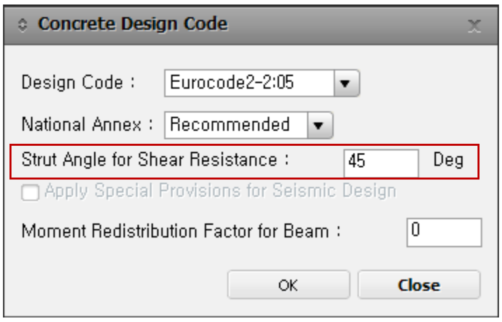
$$V_{Rd} = V_{Rd,c} \tag{5.26}$$

EN1992-1-1:2004 6.2.1(3)

Shear reinforcement

Angel between concrete compression strut and beam axis, θ, is entered in Concrete Design Code dialog box.

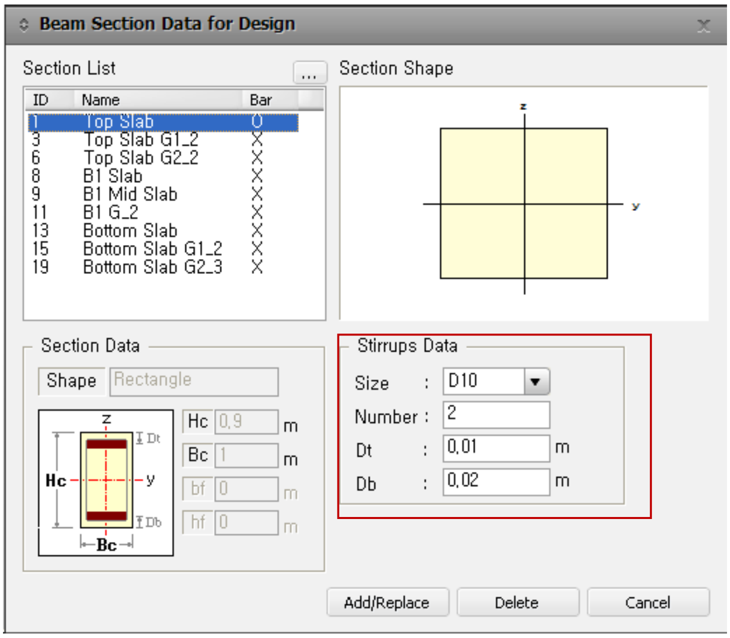
Design>RC Design> Design Code ...



[Fig. 5.16] Input shear reinforcement

Stirrup data is entered in Beam Section Data for Design dialog box.

Design>RC Design> Beam Section for Design ...



[Fig. 5.17] Input shear reinforcement

In midas Civil, the angle between shear reinforcement and the beam axis is always applied as 90 degree.

where,

Size : Diameter of shear reinforcement

Number : Leg number of shear reinforcement

Dt : Distance from the center of top rebar to top fiber of the cross section

Db : Distance from the center of bottom rebar to bottom fiber of the cross section

2.3 Check shear resistance

$$V_{Ed} \le V_{Rd} \tag{5.27}$$

where,
V_{Ed} : Design value of the applied shear force.
V_{Rd} : Design shear resistance.

• Design load combination

Load combinations used in concrete code design are generated from Results > Load combinations > Concrete Design tab. Load combinations specified as "Strength/Stress" in Active column are applied for ultimate limit state design.

2.4 Check the ratio and spacing of shear reinforcement

When no shear reinforcement is required, minimum shear reinforcement shall be applied. In this case, "s" obtained from the equation (5.28) is compared to "$s_{l,max}$" for the shear rebar verification.

EN1992-1-1:2004 (9.4),(9.5N)

$$\rho_w = \frac{A_{sw}}{s b_w \sin\alpha} \ge \rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{5.28}$$

EN1992-1-1:2004 (9.6N)

$$s \le s_{l,\max} = 0.75d(1+\cot\alpha) \tag{5.29}$$

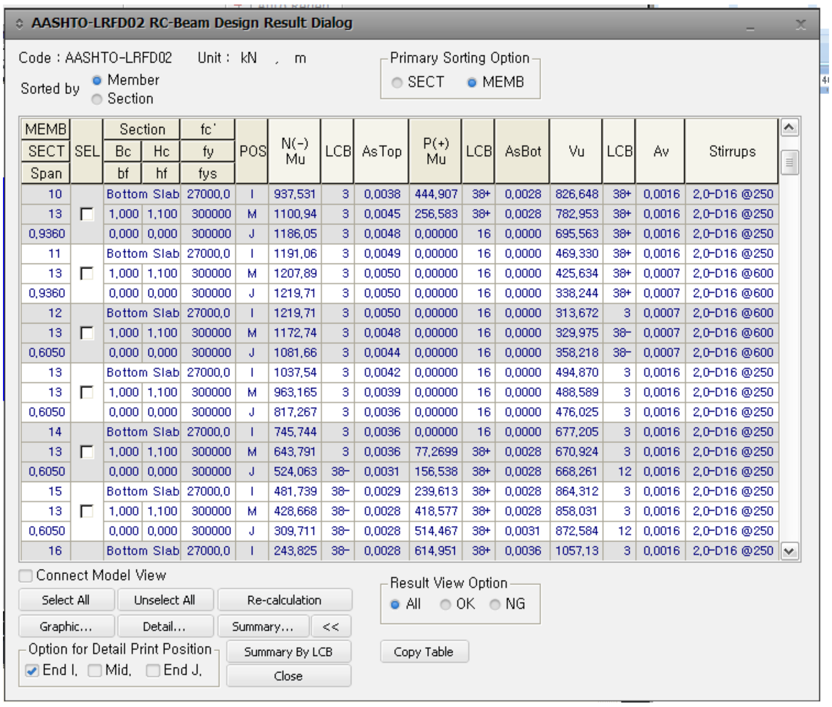
where, α is always applied as 90 degree.

3. Verification of moment and shear resistance

By Result Tables

The design results can be checked as shown in the table below.

Design>RC Design> Concrete Code Design > Beam Design ...



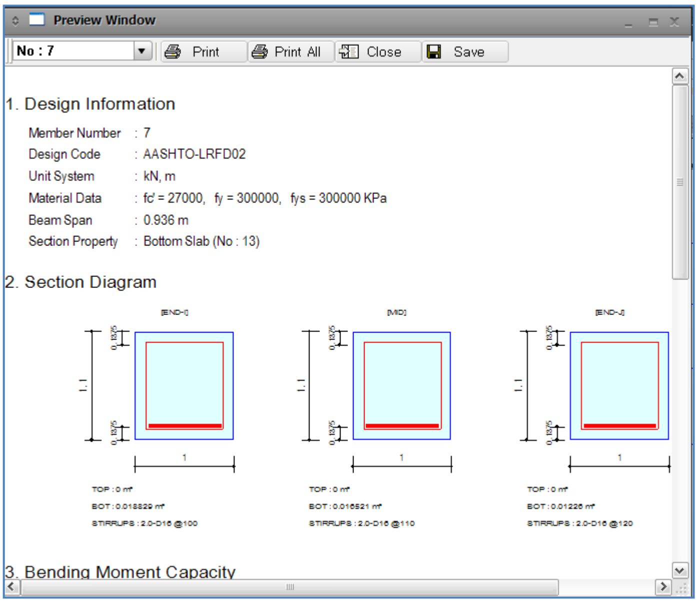
MEMB / SECT / Span	SEL	Section / Bc / bf	Section / Hc / hf	fc' / fy / fys	POS	N(-) Mu	LCB	AsTop	P(+) Mu	LCB	AsBot	Vu	LCB	Av	Stirrups
10		Bottom Slab		27000,0	I	937,531	3	0,0038	444,907	38+	0,0028	826,648	38+	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	1100,94	3	0,0045	256,583	38+	0,0028	782,953	38+	0,0016	2,0-D16 @250
0,9360		0,000	0,000	300000	J	1186,05	3	0,0048	0,00000	16	0,0000	695,563	38+	0,0016	2,0-D16 @250
11		Bottom Slab		27000,0	I	1191,06	3	0,0049	0,00000	16	0,0000	469,330	38+	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	1207,89	3	0,0050	0,00000	16	0,0000	425,634	38+	0,0007	2,0-D16 @600
0,9360		0,000	0,000	300000	J	1219,71	3	0,0050	0,00000	16	0,0000	338,244	38+	0,0007	2,0-D16 @600
12		Bottom Slab		27000,0	I	1219,71	3	0,0050	0,00000	16	0,0000	313,672	3	0,0007	2,0-D16 @600
13	☐	1,000	1,100	300000	M	1172,74	3	0,0048	0,00000	16	0,0000	329,975	38-	0,0007	2,0-D16 @600
0,6050		0,000	0,000	300000	J	1081,66	3	0,0044	0,00000	16	0,0000	358,218	38-	0,0007	2,0-D16 @600
13		Bottom Slab		27000,0	I	1037,54	3	0,0042	0,00000	16	0,0000	494,870	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	963,165	3	0,0039	0,00000	16	0,0000	488,589	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	817,267	3	0,0036	0,00000	16	0,0000	476,025	3	0,0016	2,0-D16 @250
14		Bottom Slab		27000,0	I	745,744	3	0,0036	0,00000	16	0,0000	677,205	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	643,791	3	0,0036	77,2699	38+	0,0028	670,924	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	524,063	38-	0,0031	156,538	38+	0,0028	668,261	12	0,0016	2,0-D16 @250
15		Bottom Slab		27000,0	I	481,739	38-	0,0029	239,613	38+	0,0028	864,312	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	428,668	38-	0,0028	418,577	38+	0,0028	858,031	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	309,711	38-	0,0028	514,467	38+	0,0031	872,584	12	0,0016	2,0-D16 @250
16		Bottom Slab		27000,0	I	243,825	38-	0,0028	614,951	38+	0,0036	1057,13	3	0,0016	2,0-D16 @250

[Fig. 5.18] Result table for moment resistance

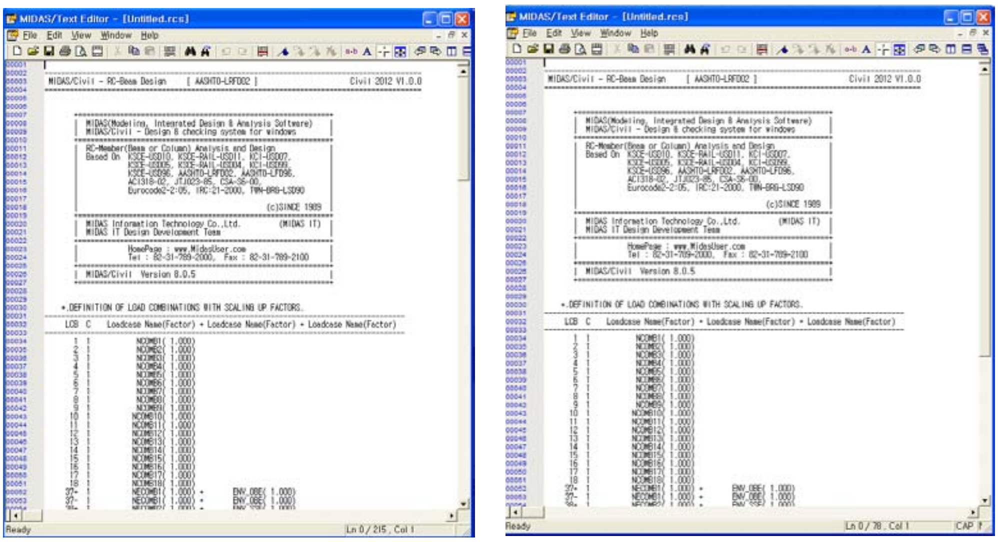
By Report

Design results can be verified in Graphic Report, Detail Report, and Summary Report.

Design>RC Design> Concrete Code Design > Beam Design ...



[Fig. 5.19] Graphic report for beam design



[Fig.5.20] Detail and Summary report for beam design

Chapter 5. Concrete Bridge Design: EN1992-2

Serviceability Limit States

1. Stress for cross section

EN1992-1-1:2004
3.1.8(1)

Stress verification will be performed for the concrete and reinforcement at the top and bottom fiber.

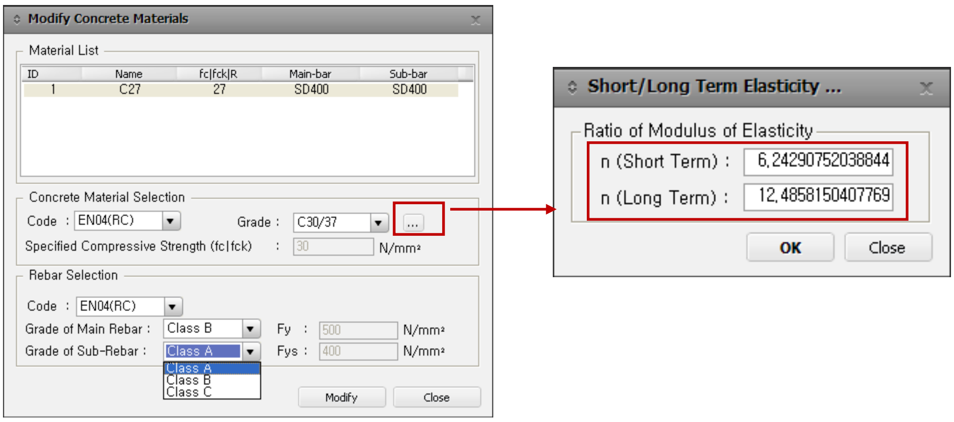
$\sigma_c \le \sigma_{ca}$, $\sigma_s \le \sigma_{sa}$

Tensile stresses in concrete and reinforcement are calculated based on the centroid in the transformed section. The ratio of modulus of elasticity in uncracked section for transformed section is entered in Modify Concrete Materials dialog box. In midas Civil, the long-term ratio, n, is applied.

When calculating stress in uncracked section, the ratio of modulus of elasticity is changed depending on the load combination. When the load combination is quasi=-permanent live load, the ratio of short term is applied.

Short/Long Term Elasticity

Design > RC Design > Modify Concrete Material



[Fig.5.21] Short/Long Term Elasticity

Default value of ratio is entered as Es/Ec for short term and 2(Es/Ec) for long term respectively. The value can be specified by the user directly.

1.1 Allowable tensile stress of concrete

$$\sigma_{ca} = \max\left(f_{ctm},\ \ (1.6 - h/1000)\, f_{ctm} \right) \tag{5.30}$$

where,

h : The total member depth

f_{ctm} *: The mean value of axial tensile strength of concrete.*

[Table5.11] Mean value of axial tensile strength, f_{ctm}

Condition	f_{ctm}
≤ C50/60	$0.30 f_{ck}^{2/3}$
> C50/60	$2.12 \ln(1+(f_{cm}/10))$

f_{cm} : The mean compressive strength at 28 days.

$$f_{cm} = f_{ck} + 8MPa \tag{5.31}$$

$$\sigma_{cm} = k_1 f_{ck} \tag{5.32}$$

where,

f_{ck} : The concrete compressive strength

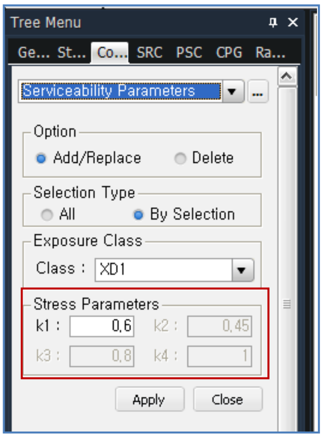
k_1,~ k_4 is applied as shown in the table below. The user can directly enter the values for k_1~k_4.

[Table5.12] Coefficient k_1~ k_4

k_1	k_2	k_3	k_4
0.6	0.45	0.8	1.0

Coefficient k_1~ k_4 for Concrete

Design>RC Design> Serviceability Parameters ...



[Fig.5.22] Input coefficient k_1~ k_4 for stress limitation

1.2 Allowable tensile stress of reinforcement

$$\sigma_{sa} = k_3 f_{yk} \tag{5.33}$$

where,

f_{yk}: The characteristic yield strength of reinforcement.

2. Crack width

Cracking shall be limited to satisfy the following condition.
Crack width, w_k ≤ Crack width limit, w_{max}

2.1 Calculate crack widths

(1) Determine ε_{sm}-ε_{cm}

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1+\alpha_e \rho_{p,eff}\right)}{E_s} \geq 0.6\frac{\sigma_s}{E_s} \quad (5.34)$$

EN1992-1-1:2004 (7.9)

where,

ε_{sm} : The mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tensile stiffening.
ε_{cm} : The mean strain in the concrete between cracks.
σ_s : The stress in the tension reinforcement assuming a cracked section.
α_e : The ratio of E_s/E_{cm}.
E_s : The design value of modulus of elasticity of reinforcing steel.
E_{cm} : The secant modulus of elasticity of concrete.(MPa)

$$E_{cm} = 22\left(\frac{f_{cm}}{10}\right)^{0.3} \quad (5.35)$$

EN1992-1-1:2004 Table 3.1

$$f_{ct,eff} = f_{ctm} \quad (5.36)$$

EN1992-1-1:2004 (7.10)

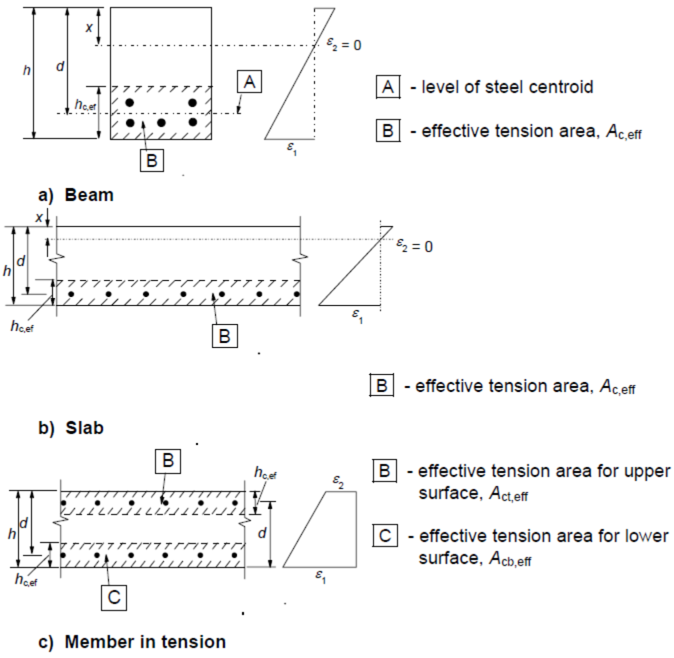
$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A_p'}{A_{c,eff}} = \frac{A_s}{A_{c,eff}} \quad (5.37)$$

A_p' : The area of pre or post-tensioned within $A_{c,eff}$. In midas Civil, A_p' is applied as zero since tendon is not considered.
$A_{c,eff}$: The effective area of concrete in tension surrounding the reinforcement of prestressing tendons of depth, $h_{c,ef}$.

$$h_{c,ef} = \min\left[2.5(h-d),\ \frac{h-x}{3},\ \frac{h}{2}\right] \quad (5.38)$$

EN1992-1-1:2004 7.3.2(3)

EN1992-1-1:2004
Figure 7.1



[Fig.5.23] Effective tension area (typical cases)

k_t : *A factor dependent on duration of the load.*

EN1992-1-1:2004
7.3.4(2)

[Table5.13] Factor k

Condition	k_t
Short term loading	0.6
Long term loading	0.4

• Definition of Short and Long term loads

[Table5.14] Definition of duration of the load

Condition	Description
Long term loading	Load combinations composed of long-term load cases only
Short term loading	Load combinations Other than long-term loading excepting for the long-term loading

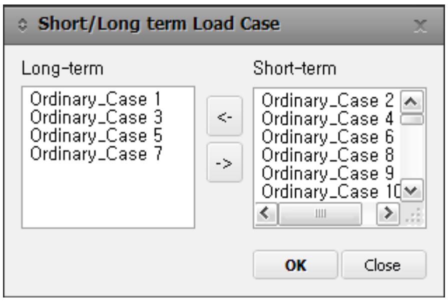
If the user does not specify the long-term or short-term load case, the load cases are classified as shown in the table below.

[Table5.15] Classification for duration of the load

Duration of the load	Description
Long term load case	Following static load case D : Dead Load DC : Dead Load of Component and Attachments. DW : Dead Load of Wearing Surfaces and Utilities. L : Live Load. LR : Roof Live Load.
Short term load case	Load cases other than long-term load cases

Duration of load (Short/Long term)

Design>Common Parameter>Short/Long term Load Case



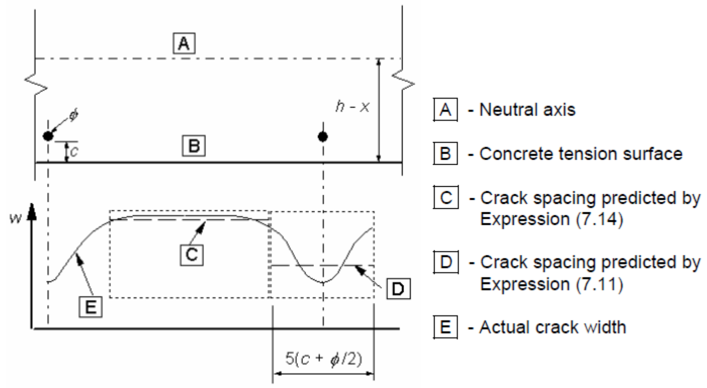
[Fig.5.24] Define short/long term load case

(2) Determine $s_{r,max}$

The maximum crack spacing, $s_{r,max}$ is calculated as shown in the table below.

$$s_{r.\max} = k_3 c + \frac{k_1 k_2 k_4 \phi}{\rho_{p,eff}} \qquad (5.39)$$

EN1992-1-1:2004 (7.11)



EN1992-1-1:2004 Figure 7.2

[Fig. 5.25] Crack width, w, at concrete surface relative to distance from bar

where,

φ : The bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, φ_{eq}, should be used.

For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2.

$$\phi_{eq} = \frac{n_1 \phi_1^2 + n_2 \phi_2^2}{n_1 \phi_1 + n_2 \phi_2} \qquad (5.40)$$

EN1992-1-1:2004 (7.12)

c : The cover to the longitudinal reinforcement.
k_1 : A coefficient which takes account of the bond properties of the bonded reinforcement (= 0.8 for high bond bars)
k_2 : A coefficient which takes account of the distribution of strain. (= 0.5 for bending)
k_3 = 3.4 (recommended values)
k_4 = 0.425(recommended values)

(3) Calculate the design crack width, w_k

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{5.41}$$

EN1992-1-1:2004 (7.8)

2.2 Get a limiting calculated crack width, w_{max}

(1) Recommended values of w_{max} (mm)

For reinforced members without prestressing tendon, a limiting crack width, w_{max}, are given in the table below.

[Table5.16] Limiting crack width, w_{max}

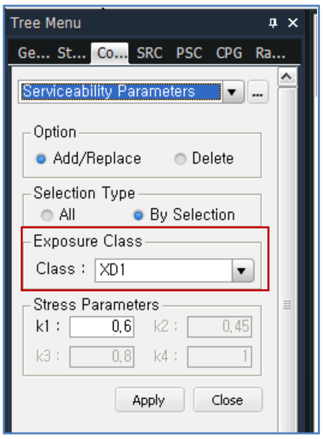
<table>
<tr><th rowspan="2">Exposure Class</th><th colspan="3">Serviceability Load combination Type</th></tr>
<tr><th>Quasi</th><th>Frequent</th><th>Characteristic</th></tr>
<tr><td>X0</td><td rowspan="2">0.4</td><td rowspan="18">User defined</td><td rowspan="11">Not Checked</td></tr>
<tr><td>XC1</td></tr>
<tr><td>XC2</td><td rowspan="3">0.3</td></tr>
<tr><td>XC3</td></tr>
<tr><td>XC4</td></tr>
<tr><td>XD1</td><td rowspan="3">0.3</td></tr>
<tr><td>XD2</td></tr>
<tr><td>XD3</td></tr>
<tr><td>XS1</td><td rowspan="3">0.3</td></tr>
<tr><td>XS2</td></tr>
<tr><td>XS3</td></tr>
<tr><td>XF1*</td><td rowspan="7">Not Checked</td><td rowspan="7">0.2</td></tr>
<tr><td>XF2*</td></tr>
<tr><td>XF3*</td></tr>
<tr><td>XF4*</td></tr>
<tr><td>XA1*</td></tr>
<tr><td>XA2*</td></tr>
<tr><td>XA3*</td></tr>
</table>

(*) For "Freeze/Thaw attack class(XF1~XF4) and Chemical attack class(XA1~XA3)", midas Civil applies the limiting crack width as 0.2mm under the characteristic load combinations.

Exposure Class

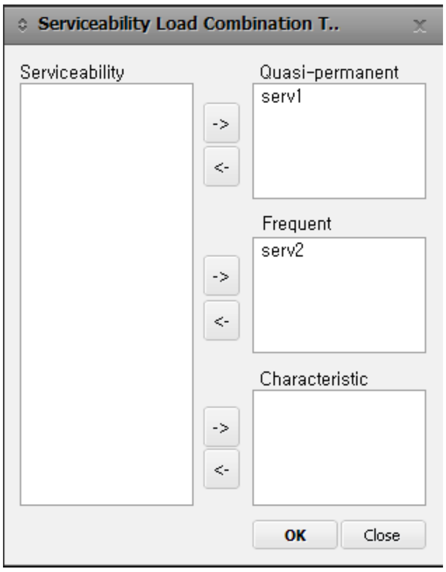
Exposure class can be defined by members in the following dialog box.

Design>RC Design> Serviceability Parameters ...



[Fig.5.26] Input Exposure class

Serviceability limit state is changed depending on the load combination type (Quasi-permanent, Frequent and Characteristic). The service load combinations can be classified in Serviceability Load Combination Type dialog box. Stress, Crack, and Deflection verifications are performed for the classified load combinations.



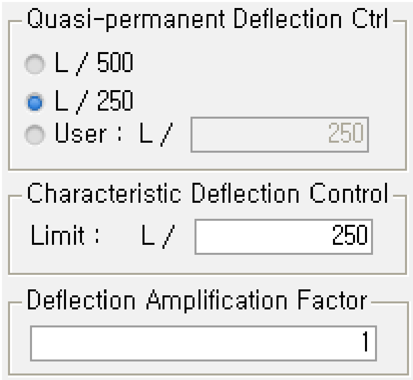
[Fig.5.27] Serviceability Load Combination

2.3 Check crack width at service loads

$$w_k \leq w_{max} \tag{5.42}$$

3. Deflection

Deflection verification is performed by comparing the deflection of the member to deflection limit. Deflection is verified for Quasi-permanent and Characteristic load combinations. The limit value is specified by the user in Serviceability Parameter dialog box.



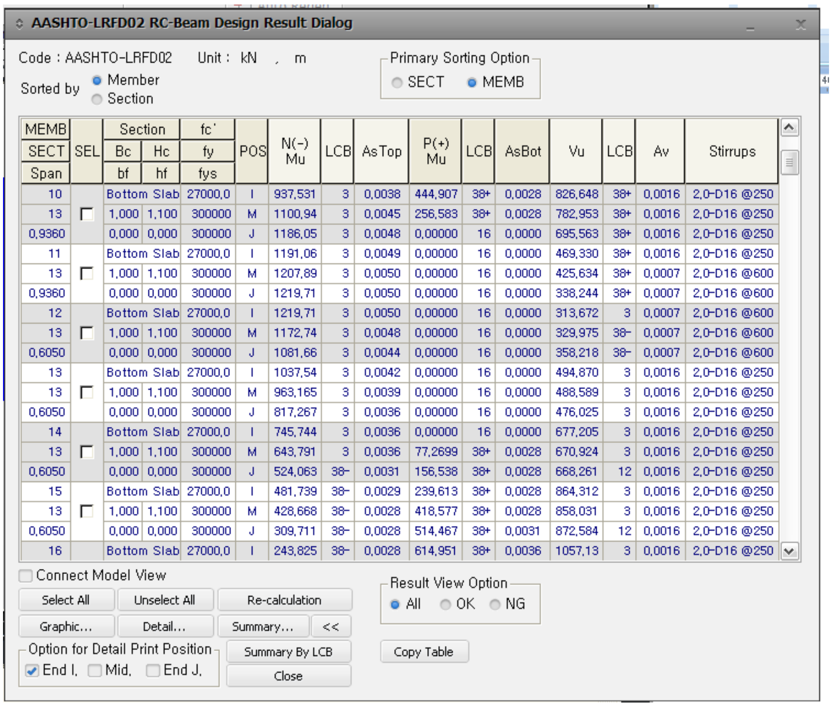
[Fig.5.28] Defection Control

4. Verification of Stress, Crack, Deflection

By Result Tables

The design results can be checked as shown in the table below.

Design>RC Design> Concrete Code Check > Beam Checking …



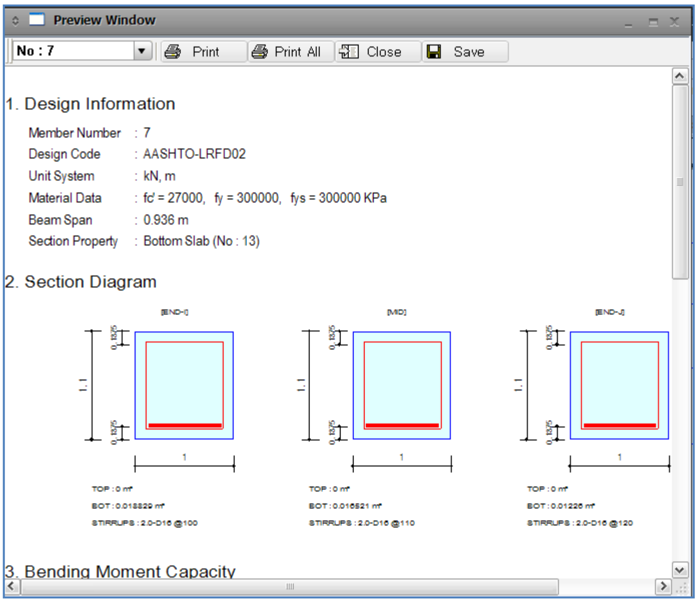
MEMB / SECT / Span	SEL	Section: Bc / bf	Hc / hf	fc' / fy / fys	POS	N(-) Mu	LCB	AsTop	P(+) Mu	LCB	AsBot	Vu	LCB	Av	Stirrups
10		Bottom Slab		27000,0	I	937,531	3	0,0038	444,907	38+	0,0028	826,648	38+	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	1100,94	3	0,0045	256,583	38+	0,0028	782,953	38+	0,0016	2,0-D16 @250
0,9360		0,000	0,000	300000	J	1186,05	3	0,0048	0,00000	16	0,0000	695,563	38+	0,0016	2,0-D16 @250
11		Bottom Slab		27000,0	I	1191,06	3	0,0049	0,00000	16	0,0000	469,330	38+	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	1207,89	3	0,0050	0,00000	16	0,0000	425,634	38+	0,0007	2,0-D16 @600
0,9360		0,000	0,000	300000	J	1219,71	3	0,0050	0,00000	16	0,0000	338,244	38+	0,0007	2,0-D16 @600
12		Bottom Slab		27000,0	I	1219,71	3	0,0050	0,00000	16	0,0000	313,672	3	0,0007	2,0-D16 @600
13	☐	1,000	1,100	300000	M	1172,74	3	0,0048	0,00000	16	0,0000	329,975	38-	0,0007	2,0-D16 @600
0,6050		0,000	0,000	300000	J	1081,66	3	0,0044	0,00000	16	0,0000	358,218	38-	0,0007	2,0-D16 @600
13		Bottom Slab		27000,0	I	1037,54	3	0,0042	0,00000	16	0,0000	494,870	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	963,165	3	0,0039	0,00000	16	0,0000	488,589	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	817,267	3	0,0036	0,00000	16	0,0000	476,025	3	0,0016	2,0-D16 @250
14		Bottom Slab		27000,0	I	745,744	3	0,0036	0,00000	16	0,0000	677,205	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	643,791	3	0,0036	77,2699	38+	0,0028	670,924	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	524,063	38-	0,0031	156,538	38+	0,0028	668,261	12	0,0016	2,0-D16 @250
15		Bottom Slab		27000,0	I	481,739	38-	0,0029	239,613	38+	0,0028	864,312	3	0,0016	2,0-D16 @250
13	☐	1,000	1,100	300000	M	428,668	38-	0,0028	418,577	38+	0,0028	858,031	3	0,0016	2,0-D16 @250
0,6050		0,000	0,000	300000	J	309,711	38-	0,0028	514,467	38+	0,0031	872,584	12	0,0016	2,0-D16 @250
16		Bottom Slab		27000,0	I	243,825	38-	0,0028	614,951	38+	0,0036	1057,13	3	0,0016	2,0-D16 @250

[Fig.5.29] Result table for moment resistance

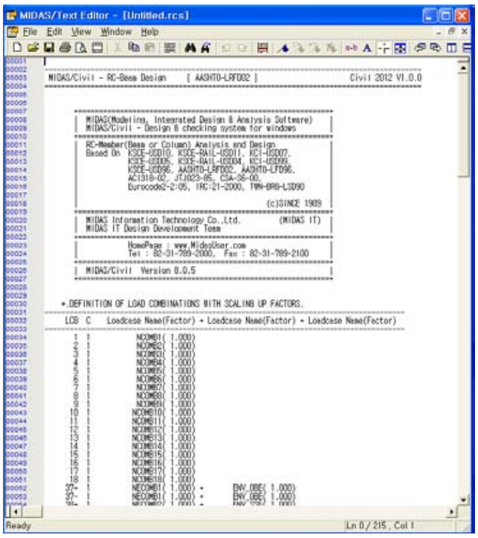
By Report

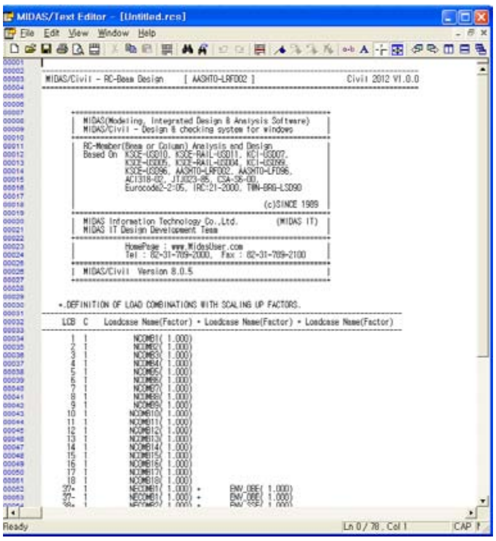
Design results can be verified in Graphic Report, Detail Report and Summary Report.

Design>RC Design> Code Check > Beam Checking ...



[Fig.5.30] Graphic report for beam design





[Fig.5.31] Detail and Summary report for beam design

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