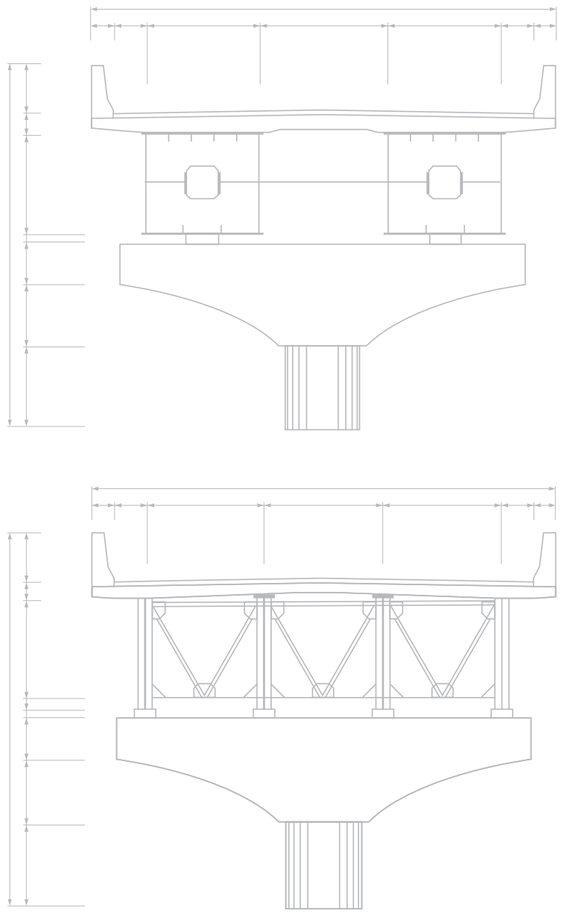
AS/NZS 5100.6-2017

DESIGN GUIDE
for midas Civil



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DESIGN GUIDE for midas Civil

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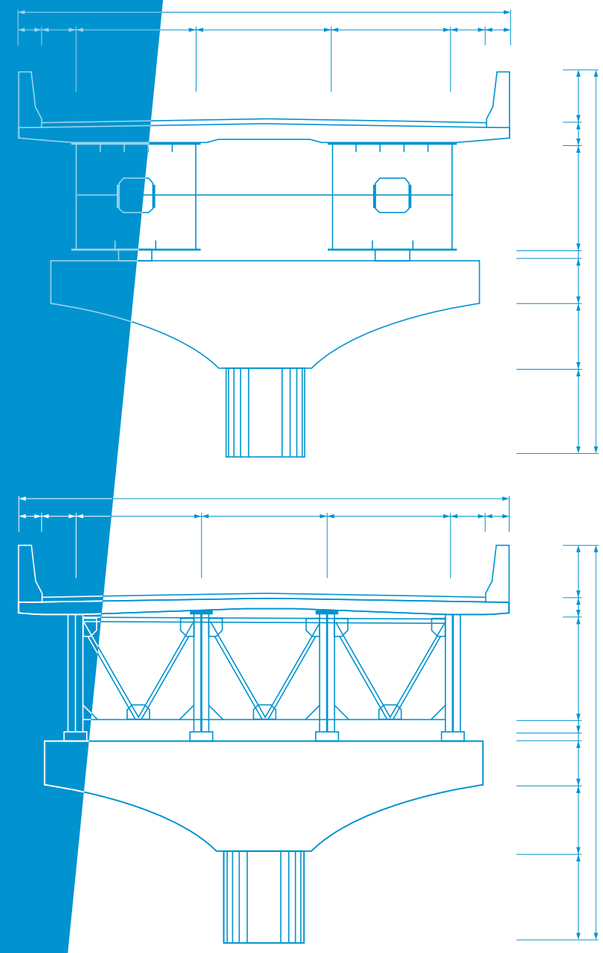
AS/NZS 5100.6-2017

DESIGN GUIDE
for midas Civil

Steel Composite
Girder Design



MIDAS

Design Guide

for MIDAS Civil

Steel Composite Girder Design

to AS/NZS 5100.6-2017

April 2024

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Introduction and Design Information

The objective of this design guide is to outline the design algorithms applied in midas Civil finite element analysis and design system for the design of composite beams as per AS5100.6:2017. The requirements of the other parts of the AS(AS/NZS) 5100 series may apply to both steel and composite members.

This chapter discusses the general requirements for the analysis and design of the steel composite sections.

1.1 Section Data

In a composite beam, a complete connection between the steel girder and the concrete deck is assumed, implying that the shear capacity of the shear connectors does not govern the maximum compressive force developed in concrete.

The scope of the design check carried by the midas Civil according to AS5100.6:2017 covers:

i) I-section

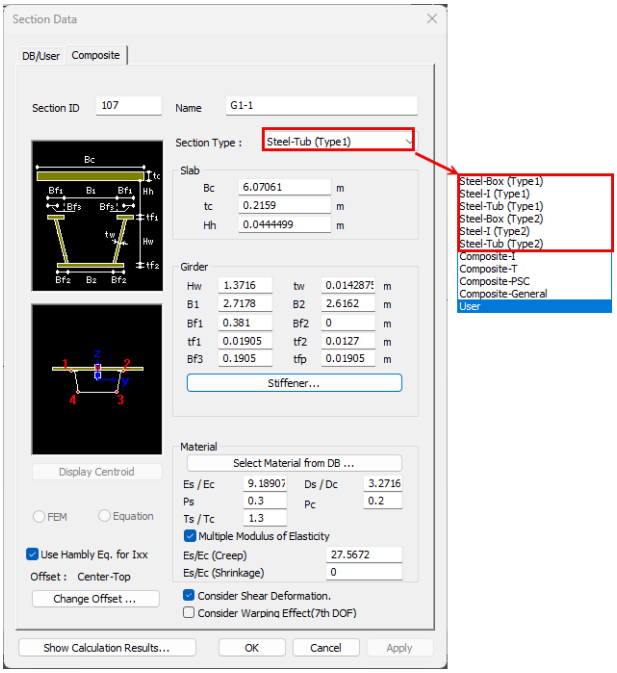
ii) Tub-type section

iii) Box-type section

Section data

In midas Civil, the section for steel composite design shall be specified from the *Composite Section Data* tab. Various sections such as Steel Box (type 1), Steel Box (type 2), Steel I (type 1), Steel I (type 2), Steel Tub (type 1), Steel Tub (type 2) are available in the database.

Properties > Section Properties > Composite



[Fig. 1.1] Steel Composite Section Database

1.2 Stiffeners of Steel Composite Section

The program considers longitudinal stiffeners under the composite girder section definition and hence accounts for girder section stiffness. The transverse stiffeners are to be additionally specified during the design check definitions of the sections.

The following table briefly describes the stiffeners used in the steel composite section.

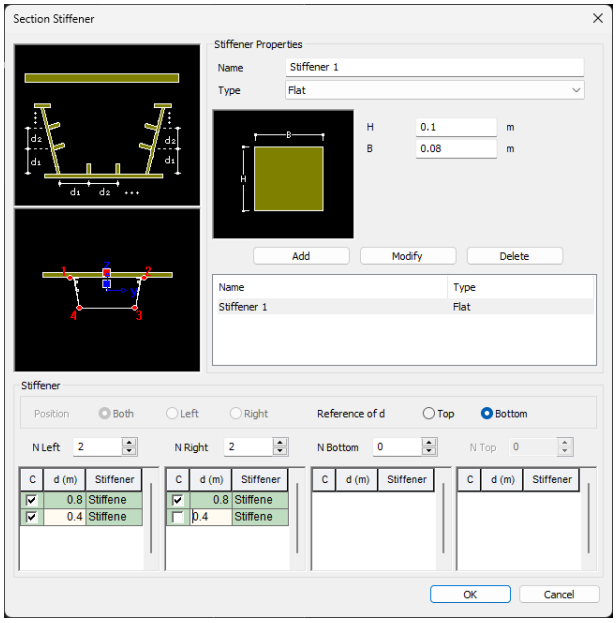
[Table 1.1] Types of Stiffeners

Type	Description
Transverse Stiffeners	Transverse stiffeners are usually provided to increase shear resistance by tension field action. These work as anchors for the tension so that post-buckling shear resistance can be developed. It should be noted that transverse stiffeners cannot prevent elastic web shear buckling.
Longitudinal Stiffeners	Longitudinal stiffeners may be provided to increase flexural resistance by preventing local buckling. These work as restraining boundaries for compression elements so that inelastic flexural buckling stress can be developed in a web. It consists of a plate, Tee section or a U-rib welded longitudinally to the web or the flange.

Longitudinal stiffener data

In midas Civil, the longitudinal stiffeners for a section shall be specified from the *Composite Section Data* tab, where the flat, Tee-shaped, and U-rib sections can be defined.

Properties > Section Properties > Composite > Stiffener...



[Fig. 1.2] Longitudinal stiffener data for section

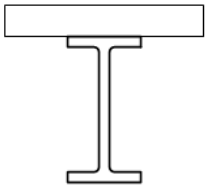
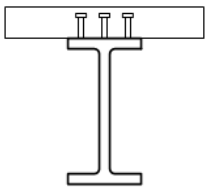
1.3 Composite Action

The composite action is generally considered when the slab is hardened, and the temporary falsework is removed. The steel girders that are compact during composite action with the slab may not be compact before composite action; thus, in that case, the section shall be checked in the stage of loading, which is not composite.

1.4 Construction Stage for Steel Composite Beam

During the construction of a steel composite bridge, the steel girder is erected first, followed by concrete deck pour and finally hardening of deck concrete. Hence, the construction of the steel composite girder is divided into three significant steps.

[Table1.3] Construction Stage for Steel Composite Section

Construction stage for steel composite section	Description	
Only Steel Girder (non-composite)	Only the steel girder has been constructed.	
Steel girder and concrete deck as load (non-composite)	Although the concrete deck has been constructed, it has not hardened yet. Therefore, the weight of the wet concrete is applied as a load condition.	
Steel girder and concrete deck as member (composite)	After the concrete is hardened, the strength and stiffness are formed. Hereafter, the steel girder and concrete deck work as a complete composite section.	

We can utilize the Construction Stages function to portray the steel composite section construction process within the program.

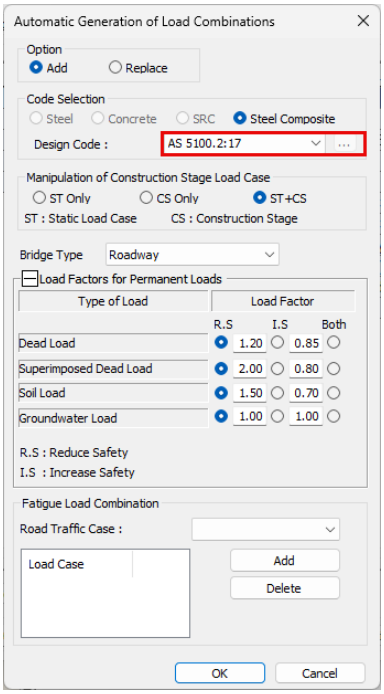
1.5 Time-Dependent Material

The steel composite section is composed of steel and concrete. Concrete has time-dependent material properties, and there is a development in the compressive strength and change of creep & shrinkage properties with time. Also, the restraints imposed by the shear connectors cause additional stresses within the composite section. Therefore, time-dependent characteristics (creep and shrinkage) must be considered.
Modular ratio is the ratio of the modulus of elasticity of steel to that of concrete. It can be specified during the section properties definition within the *Section Data* dialog box.

1.6 Load combinations

In midas Civil, the load combinations can be automatically generated for steel-composite bridge design as per AS5100.2:2017.

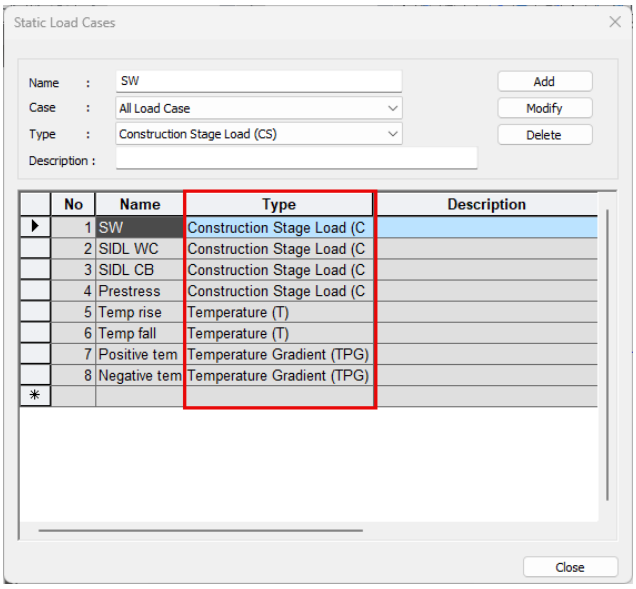
- *Results > Load Combination > Composite Steel Girder Design > Auto Generation…*



[Fig. 1.3] Auto Generation of Load Combinations

However, it should be noted that the load case type must be correctly specified to generate the automatic load combinations.

- *Load > Static Load Cases…*



[Fig. 1.4] Load case type

1.7 Design Information

Following is the list of specifications necessary for the design of steel composite members in midas Civil.

1.7.1 Design value of material

The nominal values of the yield and the ultimate strength for the structural steel shall be taken from the appropriate code depending upon the grade of steel, concrete and reinforcement chosen, as shown in the figure below:

TABLE 2.1

STRENGTHS OF STEELS COMPLYING WITH AS/NZS 1163, AS/NZS 1594, AS/NZS 3678, AS/NZS 3679.1, AS/NZS 3679.2(Note 3) AND AS 3597

Steel Standard	Form	Steel grade	Thickness of material (t) mm	Yield stress (f_y) MPa	Tensile strength (f_u) MPa
AS/NZS 1163 (Note 3)	Hollow sections	C450	All	450	500
		C350	All	350	430
		C250	All	250	320
AS/NZS 1594	Plate, strip, sheet floorplate	HA400	All	380	460
		HW350	All	340	450
		HA350	All	350	430
		HA300 HU300	All	300	400
		HA250 HU250	All	250	350
AS/NZS 3678 (Note 2 and 3)	Plate and floorplate	450	$t \le 20$	450	520
		450	$20 < t \le 32$	420	500
		450	$32 < t \le 50$	400	500
		400	$t \le 12$	400	480
		400	$12 < t \le 20$	380	480
		400	$20 < t \le 80$	360	480
		350	$t \le 12$	360	450
		350	$12 < t \le 20$	350	450
		350	$20 < t \le 80$	340	450
		350	$80 < t \le 150$	330	450
		WR350	$t \le 50$	340	450
		300	$t \le 8$	320	430
		300	$8 < t \le 12$	310	430
		300	$12 < t \le 20$	300	430
		300	$20 < t \le 50$	280	430
		300	$50 < t \le 80$	270	430
		300	$80 < t \le 150$	260	430
		250	$t \le 8$	280	410
		250	$8 < t \le 12$	260	410
		250	$12 < t \le 50$	250	410
		250	$50 < t \le 80$	240	410
		250	$80 < t \le 150$	230	410
		200	$t \le 12$	200	300

(continued)

[Fig. 1.5] Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel

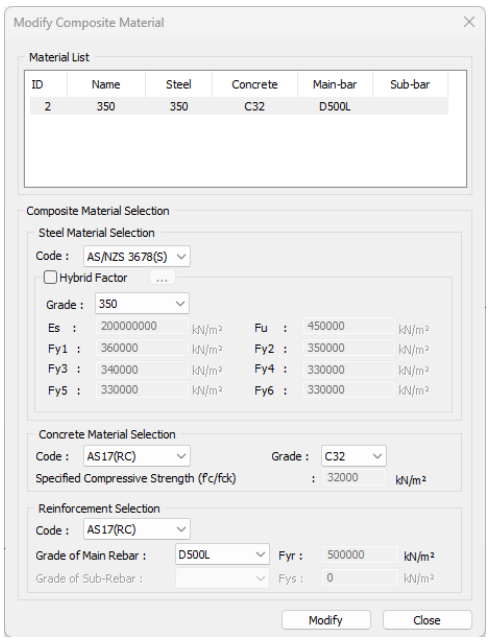
The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel shall be obtained using the table in Figure. 1.5.

☐ Design Strength of Materials

The design strength of concrete, reinforcement, and structural steel can be defined in the "Modify Composite Material" dialog box. f_y is the tensile strength of the steel, and fu is the ultimate strength of steel. Depending upon the code chosen AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 4672.1, the Tensile strength, F_{y1}, F_{y2}, etc... are automatically updated considering the given plate thickness. Similarly, the properties of the chosen grade of concrete and rebar are automatically

updated and used in the design check.

- *Design > Composite Design > Design Material*



[Fig. 1.6] Composite Steel Girder Design Material

1.7.2 Design Parameters

The general design parameters and the related design checks for ULS and SLS that are to be carried out are to be specified herewith.

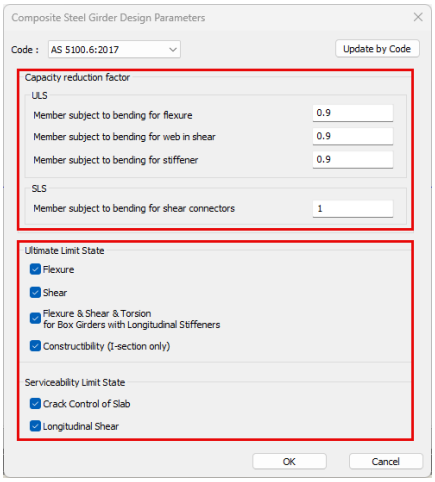
The capacity reduction factor (ϕ) considered in the ultimate (strength) limit state is taken as 0.9 as per Table 3.2 of AS5100.6:2017. It can be modified as per the user's requirement.

Similarly, we can choose what all checks under ULS and SLS are to be performed for the section in the same dialog box. Under ULS combinations, we can select flexure and shear checks. Under SLS combinations, we can check the cracking of the slab and longitudinal shear in the section.

Capacity Reduction Factors and General Design Parameters

Parameters related to the capacity reduction factors for ULS and SLS combinations can be defined in the *Composite Steel Girder Design Parameters* dialog box. The default values of partial factors are defined per code provisions of AS5100.6:2017.

Design > Composite Design > Design Parameters…

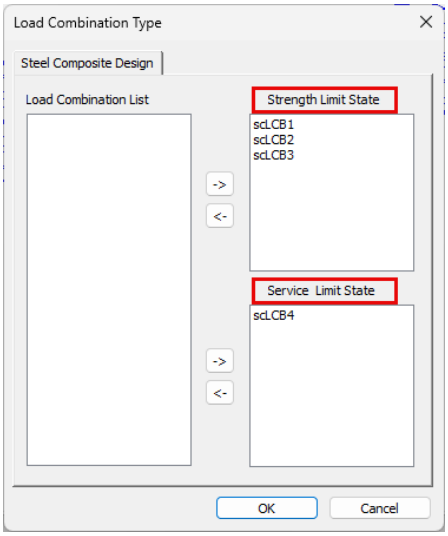


[Fig. 1.7] Design Parameters Dialog Box

1.7.3 Load Combination Type

The load combinations defined for the design of the steel composite sections are automatically subdivided into the strength and serviceability limit states for design checks. We can verify these combinations in the dialog box and make the necessary changes.

Design > Composite Design > Load Combination Type…

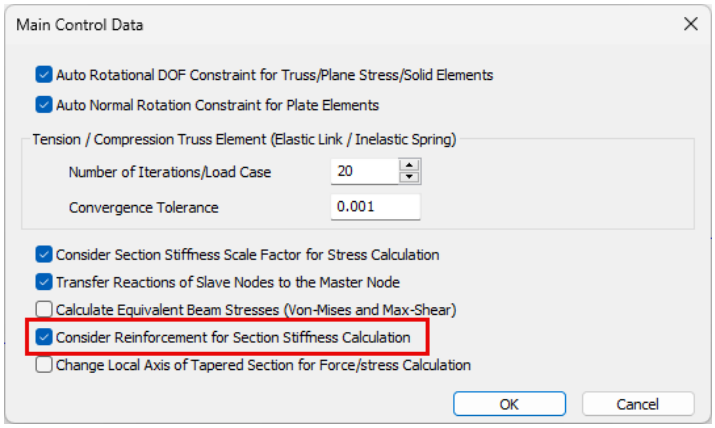


[Fig. 1.8] Load Combination Type Dialog Box

1.7.4 Longitudinal Reinforcement

The provision of longitudinal reinforcement in the section under consideration is necessary for design checks. Longitudinal reinforcement may or may not be considered in the stiffness calculation section, depending on the option chosen in the *Main Control Data* tab.

Analysis > Main Control Data…

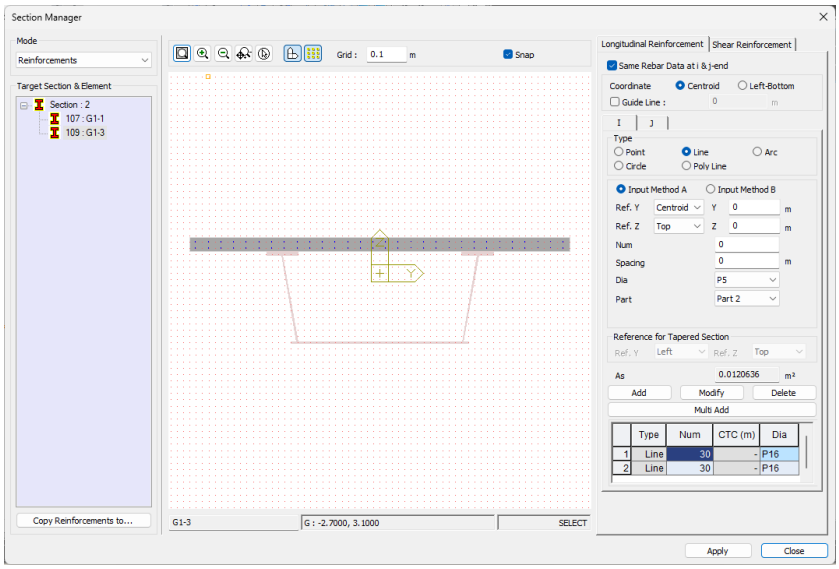


[Fig. 1.9] Main Control Data Dialog Box

Longitudinal reinforcement

Longitudinal reinforcement can be provided in the slab section ó́f the composite beam by using either of the methods available, Method A or Method B.

Design > Composite Design > Longitudinal Reinforcement…



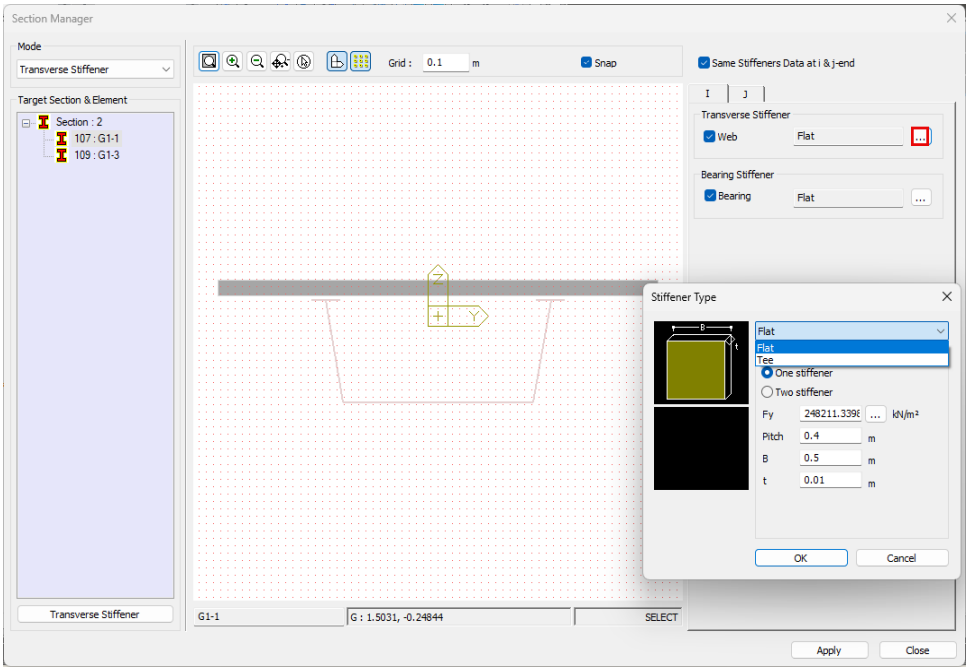
[Fig. 1.10] Input for Longitudinal Reinforcement

1.7.5 Transverse Stiffeners

Transverse Stiffener

Transverse Web stiffeners can be specified under the *Design tab.*

Design > Composite Design > Transverse Stiffener…



[Fig. 1.11] Transverse Stiffener for Design Data Input

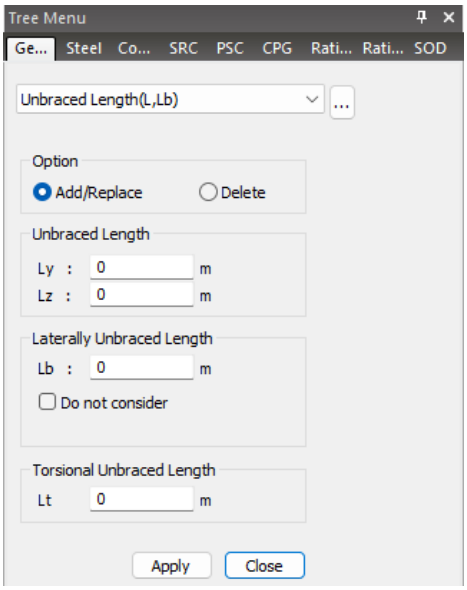
1.7.6 Unbraced Length of the Beam

To accurately calculate the beam capacity under flexure for the section under consideration, the unbraced length parameter shall be specified. These parameters shall be used to calculate lateral torsional buckling.

Unbraced Length(L,Lb)

Parameters related to the beam length used in calculating beam section capacity can be defined in the *"Common Parameters"* dialog box or in the "*Composite Design*" tab.

Design > Composite Design > Unbraced length (L, Lb)…



[Fig. 1.12] Unbraced length

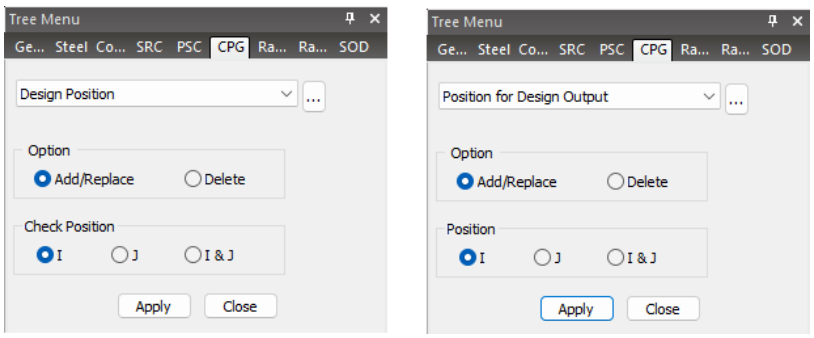
1.7.7 Design Positions

It is often desired to design only the critical element of the girder, or only the critical positions of the section are to be considered in the design. Thus, to avoid designing the complete structure, the design check can be carried out more efficiently by specifying the required sections.

Design Position and Position for Design Output

Select the element to be designed and specify the section position (*i* or *j or both*) for the same. Sections thus selected shall be designed. Only the sections considered in the *Position for Design Output* shall be shown in the output results.

Design > Composite Design > Design Positions / Positions for Design Output…



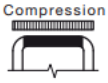
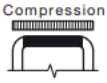
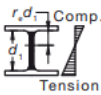
[Fig. 1.13] Design Positions and Positions for Design Output

1.7.8 Section Slenderness

The section plasticity and yield slenderness limit shall be determined according to the residual stress type.

TABLE 5.1.2

VALUES OF PLATE ELEMENT SLENDERNESS LIMITS

Plate element type	Longitudinal edges supported	Residual stress (see Notes)	Plasticity limit (λ_{ep})	Stress distribution	Yield limit (λ_{ey})	Stress distribution	Deformation limit (λ_{ed})
Flat	One	SR HR LW, CF HW	10 9 8 8	Compression	16 16 15 14	Compression	35 35 35 35
(Uniform compression)							
Flat	One	SR HR LW, CF HW	10 9 8 8		25 25 22 22		— — — —
(Maximum compression at unsupported edge, zero stress or tension at supported edge)							
Flat	Both	SR HR LW, CF HW	30 30 30 30	Compression 	45 45 40 35	Compression 	90 90 90 90
(Uniform compression)							
Flat	Both	Any	82		115		—
(Compression at one edge, tension at the other, neutral axis at mid height)							
Flat	Both	Any	For $1.0 \geq r_p \geq 0.5$ $\frac{111}{4.7r_p - 1}$ For $r_p < 0.5$ $\frac{41}{r_p}$		For $1.0 \geq r_e \geq 0$ $\frac{60}{r_e}$		
(Web of beam with neutral axis not at mid height)							
Circular hollow sections		SR HR LW, CF HW	50 50 42 42		120 120 120 120		— — — —

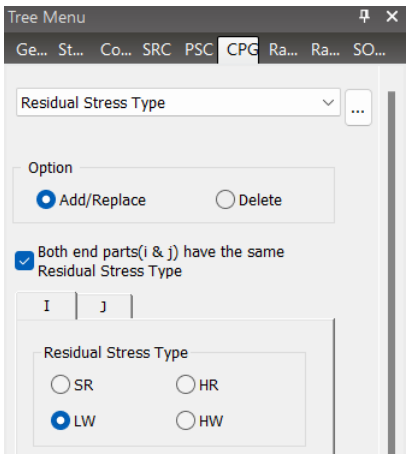
LEGEND:
SR = stress relieved
HR = hot-rolled or hot-finished
CF = cold-formed
LW = lightly welded longitudinally
HW = heavily welded longitudinally
r_p = ratio of the distance from the plastic neutral axis to the compression edge of the web to the depth of the web
r_e = ratio of the distance from the elastic neutral axis to the compression edge of the web to the depth of the web

[Fig. 1.14] Plasticity and Yield limit as per Residual stress type

Residual Stress Type

Select the elements and assign Residual Stress Type from under the Design tab.

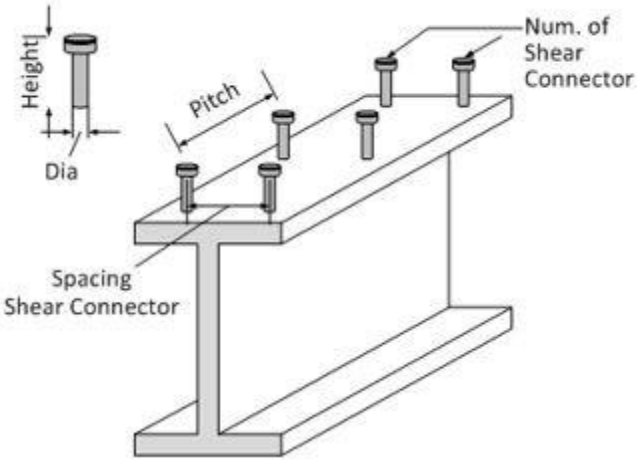
- *Design > Composite Design > Residual Stress Type…*



[Fig. 1.15] Residual Stress Type

1.7.9 Shear Connector

In Midas Civil, shear connectors should be inputted by elements. Elements having different shear connectors should be inputted separately.

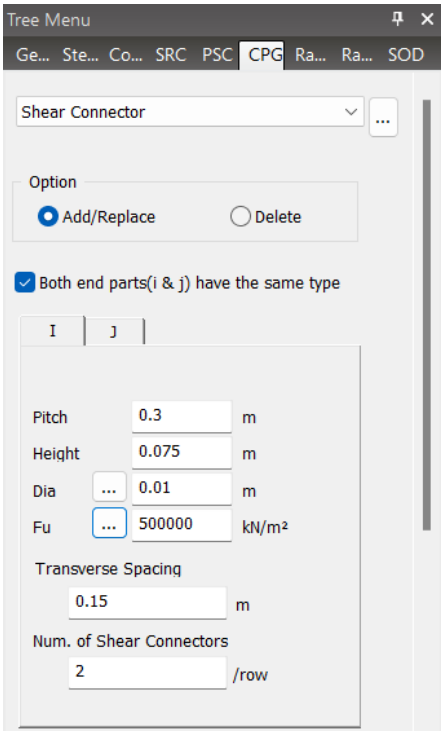


[Fig. 1.16] Shear Connector Detail

Shear Connector

Select the elements to assign shear connectors and enter the required data in the dialogue box.

Design > Composite Design > Shear Connector…



[Fig. 1.17] Shear Connector Parameters

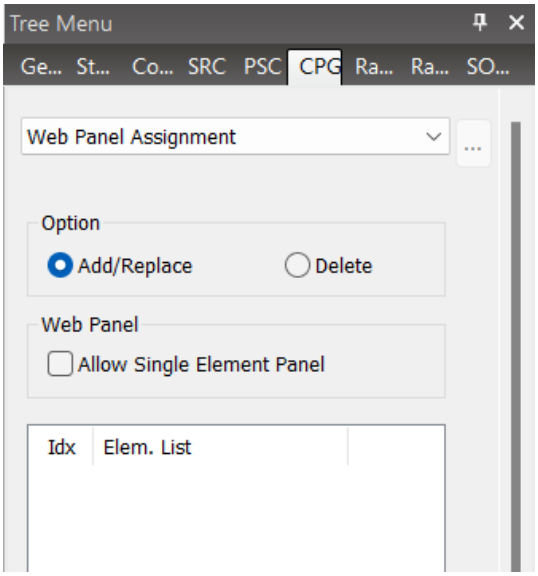
1.7.10 Buckling of Web Panel

In Midas Civil, a web panel may consist of one or more element. The buckling of these web panels is checked according to cl. 7.4.3.4.

Web Panel Assignment

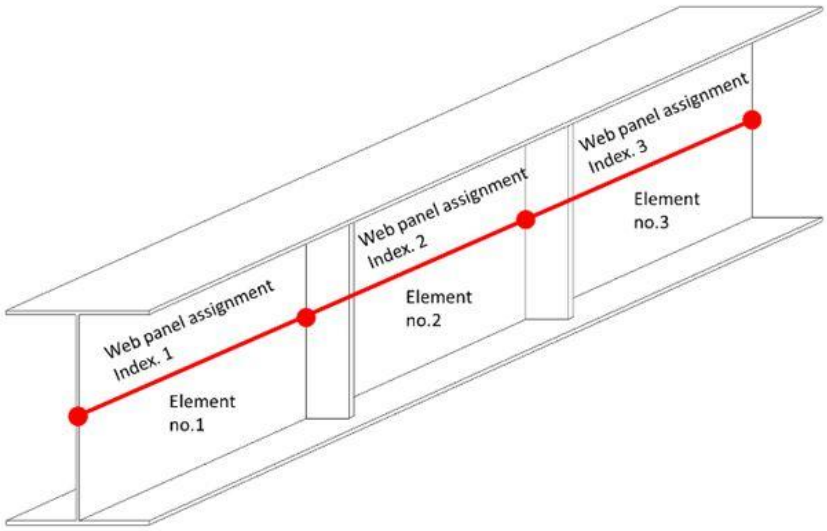
Assign a Web Panel to the selected elements.

Design > Composite Design > Web Panel Assignment…

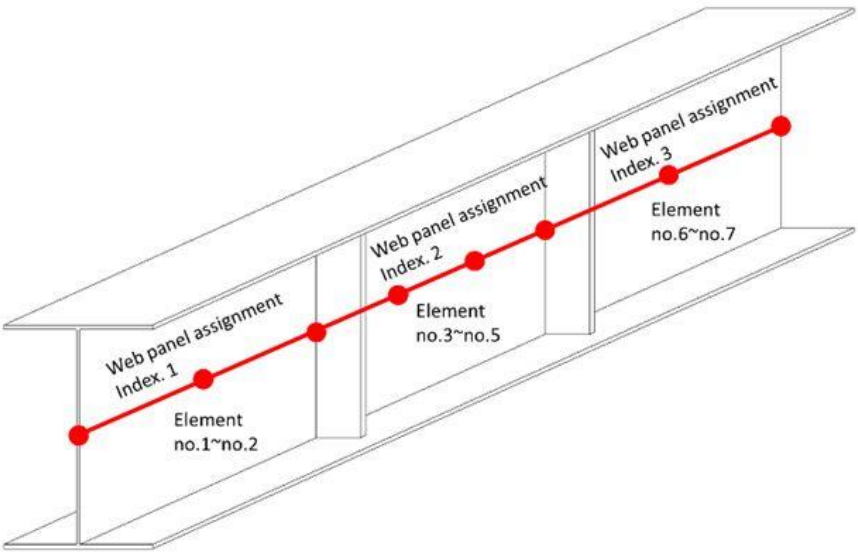


[Fig. 1.18] Web Panel Assignment

'*Allow Single Element Panel*' option can be checked on for the web panels to be located at ends of each element.



[Fig. 1.19(a)] 'Allow Single Element Panel' option is applied



[Fig. 1.19(b)] Two or three elements are assigned as web panel

Ultimate Limit State

This chapter covers the Ultimate Limit State (ULS) checks for steel composite girder design. The limit state for flexural and shear resistance is checked for the members selected for design. The checks are performed based on whether the section is an I-section, a Tub section, or a Box section. The sections are first classified as compact and not compact, and the corresponding effective section is thus calculated. The appropriate panel checks are considered depending on the number of longitudinal stiffeners provided in the section.

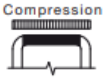
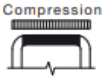
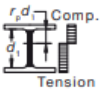
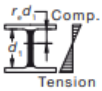
2.1 Classification of Cross Section

The cross-section of the steel beam is classified as either compact or not compact in accordance with AS5100.6:2017 using the steel section slenderness values (λs), and it is compared with the steel section plasticity slenderness limits λsp. λsp will be determined from Table 5.1.2 of AS5100.6:2017 as shown in figure 2.1 below:

AS5100.6:2017 Cl 6.3.1

AS5100.6:2017 Cl 5.1.2

TABLE 5.1.2
VALUES OF PLATE ELEMENT SLENDERNESS LIMITS

Plate element type	Longitudinal edges supported	Residual stress (see Notes)	Plasticity limit (λ_{ep})	Stress distribution	Yield limit (λ_y)	Stress distribution	Deformation limit (λ_{ed})
Flat (Uniform compression)	One	SR HR LW, CF HW	10 9 8 8	Compression	16 16 15 14	Compression	35 35 35 35
Flat (Maximum compression at unsupported edge, zero stress or tension at supported edge)	One	SR HR LW, CF HW	10 9 8 8		25 25 22 22		— — — —
Flat (Uniform compression)	Both	SR HR LW, CF HW	30 30 30 30	Compression 	45 45 40 35	Compression 	90 90 90 90
Flat (Compression at one edge, tension at the other, neutral axis at mid height)	Both	Any	82		115		—
Flat (Web of beam with neutral axis not at mid height)	Both	Any	For $1.0 \geq r_p \geq 0.5$ $\frac{111}{4.7r_p - 1}$ For $r_p < 0.5$ $\frac{41}{r_p}$		For $1.0 \geq r_e \geq 0$ $\frac{60}{r_e}$		
Circular hollow sections		SR HR LW, CF HW	50 50 42 42		120 120 120 120		— — — —

LEGEND:
SR = stress relieved
HR = hot-rolled or hot-finished
CF = cold-formed
LW = lightly welded longitudinally
HW = heavily welded longitudinally
r_p = ratio of the distance from the plastic neutral axis to the compression edge of the web to the depth of the web
r_e = ratio of the distance from the elastic neutral axis to the compression edge of the web to the depth of the web

NOTE: Welded members whose compressive residual stresses are less than 40 MPa may be considered to be lightly welded.

[Fig.2.1] Values of Plate Element Slenderness Limits

For a section with flat compression plate elements, the section slenderness (λ_s) will be taken as the value of plate element slenderness (λ_e) for the element of cross-section, which has the greatest value of $\frac{\lambda_e}{\lambda_{ey}}$

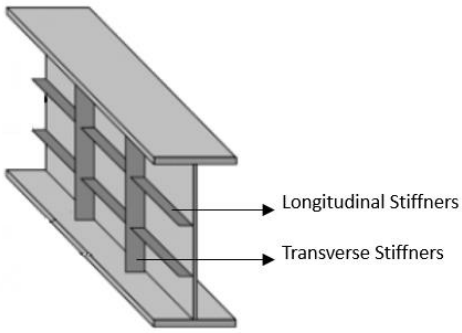
λe can be computed as follows:

$$\lambda_e = \frac{b}{t}\sqrt{\frac{f_y}{250}}$$

where,

λey = plate element yield slenderness limit as per Figure 2.1

b = clear width of element outstands as indicated in Figure 2.1 or the clear width of the element between the faces of supporting plate elements. If the longitudinal stiffeners are defined, they shall be considered as supporting plate elements to calculate b.



[Fig.2.2] Longitudinal Stiffener

The section plasticity and yield slenderness limits (λsp) and (λsy) respectively shall be taken as the values of the element plasticity and yield slenderness limits (λep) and (λey) respectively, given in Figure 2.1 for the element of the cross-section that has the greatest value of λe/λey.

(1) Compact composite section

Steel section satisfying $\lambda s \leq \lambda sp$ at the relevant loading stage is considered compact.

AS5100.6:2017 Cl 6.2.6

In sagging moment regions, the compression flange plate elements for the steel section need not satisfy $\lambda s \leq \lambda sp$ provided -

(a) the spacing of shear connectors perpendicular to the direction of compression is not greater than -

$28t_f\sqrt{\frac{250}{f_{yf}}}$; and

(b) the spacing of shear connectors in the direction of compression is not greater than

i) $14t_f\sqrt{\frac{250}{f_{yf}}}$ for unstaggered connectors or

ii) $21t_f\sqrt{\frac{250}{f_{yf}}}$ for staggered connectors

where,

tf = thickness of the compression flange plate

fyf = nominal yield stress of the flange

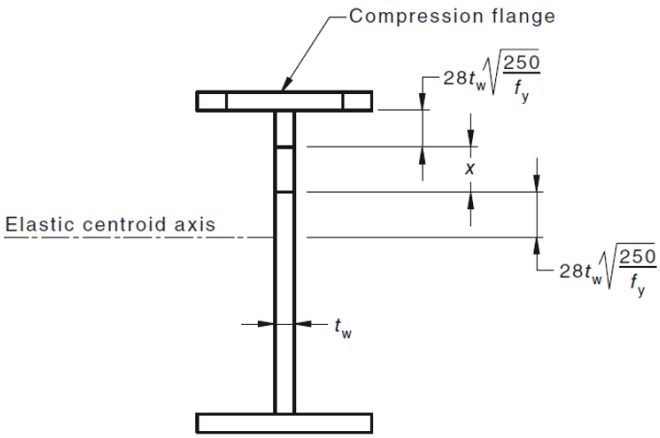
(2) Not compact composite section

A composite cross-section for which the steel section satisfies $\lambda s > \lambda sp$ at the relevant loading stage is considered as not compact.

When the web of the I-section of a channel section member is not compact, then the

effective section of the web resisting the demand shall be determined by ignoring the web section under compression corresponding to the length x, as shown in Figure 2.3 below.

AS5100.6:2017
Cl 5.1.4



[Fig.2.3] Effective portion of the steel beam with a non-compact web

The cross-section is classified based on the least favorable class of the compression part, i.e. if either the web or flange or both are classified as not compact, then the whole section is classified as not compact, else it is classified as compact.

2.2 Consideration under Moment

For the section under the sagging moment region, it is considered that compressive force in the concrete does not exceed the longitudinal shear force that can be transferred by the shear connection between the steel beam and the concrete deck slab at the strength limit state, and thus, the deck is effective.

AS5100.6:2017
Cl 6.4.1

For the section under the hogging moment region, suitably anchored deck slab reinforcement parallel to the steel section within the design effective width of the concrete deck slab will be included in computing the properties of the composite section, and the concrete deck will be completely ignored.

AS5100.6:2017
Cl 6.4.2

2.3 Compact Sections

A compact composite beam bent about the section's major principal axis shall satisfy both:

AS5100.6:2017
Cl 6.3.1

a) $M^* \leq \phi Ms$; and

b) $M^* \leq \phi Mb$

where,

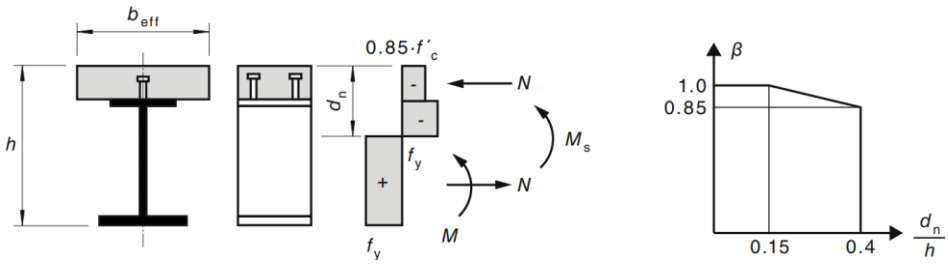
ϕ = capacity reduction factor

Ms = nominal section moment capacity

Mb = nominal beam moment capacity

When using high-strength steel (450 < fy ≤ 690 MPa), a reduction in the moment capacity to allow partial depth yielding of the beam shall be considered according to Figure 2.4. The obtained value of β shall be multiplied by the moment capacity calculated per equations 2.3 (a) & (b).

AS5100.6:2017
Cl 6.3.3



[Fig. 2.4] Reduction in Moment Capacity for Use of High-Strength Steel

2.3.1 Section Moment Capacity

For a compact section, the nominal section moment capacity (Ms) of a compact cross-section in sagging or hogging bending is determined as:

AS5100.6: 2017 Cl 6.4.3

$$Ms = Mp$$

where,

Mp = nominal section moment capacity determined by the simple plastic theory.

2.3.2 Beam Moment Capacity

For the beam continuously restrained by the deck at the compression flange level, the nominal beam moment capacity (Mb) shall be taken as:

AS5100.6: 2017 Cl 6.5.1

Mb = Ms

where,

Ms = nominal section moment capacity of the critical section determined as per section 2.4.1

2.4 Not Compact Sections

A composite beam classified as not compact, bent about the section's major principal axis, shall satisfy:

AS5100.6: 2017 Cl 6.3.2

a) $f_s^* \leq \phi f_y$;

b) $f_s^* \leq \phi f_b$; and

c) $f_c^* \leq 0.62 f_c^{'}$

where,

fs* = summation of stress in steel flanges at the stage of construction under consideration

fc* = concrete compressive stress in the sagging moment regions

Each of the criteria shall be checked as follows:

2.4.1 Flange Yielding Stress

Considering each stage of construction, the total flange stress (fs*) for each flange shall be checked in accordance with the expression shown below:

$$f_s^* \leq \phi f_y$$

The total stress developed in a flange (for the transformed section) is checked without considering the effect of the slab if the slab is in tension.

2.4.2 Flange Buckling Stress

For checking the buckling of the flange, the stress in the flange is calculated in accordance with section 2.5.

$$f_s^* \leq \phi f_b$$

fb, which is determined for the highest stressed flange, shall be obtained by using the following expression:

$$f_b = \frac{M_b}{Z_{enc}} \leq f_y$$

For determining the value of Mb for the not compact section with the beam continuously restrained by the deck not at the compression flange level, the following expression shall be considered:

AS5100.6: 2017
Cl 6.5.2

$$M_b = M_{bs}\left(\frac{Z_{ec}}{Z_{es}}\right)$$

where,

Mbs = beam moment capacity (Mb) of the steel beam alone

Zec = effective section modulus of the transformed composite section (for compact sections, Zec equals Zp).

Zes = effective section modulus of the steel section alone

Under lateral torsional buckling, when restraint to the steel compression flange is provided by a deck slab connected to an unstiffened web, either directly or via the tension flange over the length of the beam, the nominal beam moment capacity (Mb) shall be the same as above.

2.4.2.1 Calculations for Mb

a) Segments fully or partially restrained at both ends

AS5100.6: 2017
Cl 5.6.1

For open sections with equal flanges and uniform cross-sections

$$M_b = \alpha_m \alpha_s M_s \leq M_s$$

where,

Ms = nominal section moment capacity of the section

The nominal section capacity of the non-compact section shall be calculated as follows:

$$M_s = f_y Z_{enc}$$

Zenc = effective elastic section modulus of the transformed steel section, which is calculated at the extreme fibers of the steel section in accordance with Figure 2.3.

Ms is calculated for both steel flanges, and the smaller value is taken.

αm = moment modification factor for bending

The moment modification factor shall be calculated as:

$$\alpha_m = \frac{1.7M_m^*}{\sqrt{[(M_2^*)^2 + (M_3^*)^2 + (M_4^*)^2]}} \leq 2.5$$

where,

$M_m^*, M_2^*, M_4^*, M_3^*$ = maximum design bending moment in the segment, design bending moment at the quarter points (2,4) of the segment and midpoint of the segment, respectively

αs = slenderness reduction factor

The slenderness reduction factor shall be calculated as follows:

$$\alpha_s = 0.6\left\{\sqrt{\left(\frac{M_s}{M_{oa}}\right)^2 + 3} - \left(\frac{M_s}{M_{oa}}\right)\right\}$$

where,

Moa = Mo, where Mo is the reference elastic buckling moment.

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{l_e^2}\right)\left[GJ + \left(\frac{\pi^2 EI_w}{l_e^2}\right)\right]}$$

where,

E, G = elastic moduli

Iy, J, Iw = section constants

le = effective length

For I-sections with unequal flanges

The nominal member moment capacity (Mb) shall be determined in accordance with 2.4.2.1(a), except that the reference buckling moment (Mo) shall be determined by the following:

AS5100.6: 2017
Cl 5.6.1.2

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{l_e^2}\right)}\left\{\sqrt{\left[GJ + \left(\frac{\pi^2 EI_w}{l_e^2}\right) + \left(\frac{\beta_x^2}{4}\frac{\pi^2 EI_y}{l_e^2}\right)\right]} + \frac{\beta_x}{2}\sqrt{\left(\frac{\pi^2 EI_y}{l_e^2}\right)}\right\}$$

The monosymmetry section constant (β_x) shall be determined using the following:

$$\beta_x = 0.8d_f\left[\left(\frac{2I_{cy}}{I_y}\right) - 1\right]$$

where,

d_f = distance between flange centroids

I_{cy} = second moment of area of the compression flange about the section principal y-axis

b) Segment unrestrained at one end

AS5100.6: 2017
Cl 5.6.2

The nominal member capacity of a segment unrestrained at one end shall be computed as:

$$M_b = \alpha_s M_s \leq M_s$$

Taking the value of αs

$$\alpha_s = 0.6\left\{\sqrt{\left(\frac{M_s}{M_{ob}}\right)^2 + 3} - \left(\frac{M_s}{M_{ob}}\right)\right\}$$

where,

Mob = elastic buckling moment

2.4.3 Concrete Stresses

For the section under sagging moment only, the concrete stress check shall be performed in accordance with the expression provided below:

$$f_c^* \leq 0.62f_c^{'}$$

where,

fc* = maximum concrete compressive stress in the slab

2.5 Strength of Stiffened Flanges

The flange plates are checked for the following stress criteria:

AS5100.6: 2017
Cl 7.3.3.1

$$\sqrt{(f_f^*)^2 + (f_2^*)^2 - (f_f^* f_2^*) + 3(f_{vf}^*)^2} \leq \phi f_{yf}$$

where,

ff*= longitudinal stress at the mid-plane of the flange plate (taken as positive in compression)

f2*= co-existent in-plane transverse stress at the mid-plane of the flange plate (assumed as zero, as it cannot be considered through 1D analysis)

fvf*= shear stress in the flange due to torsion and shear force

$= f_{vt}^* + \left(0.5f_{vj}^*\right)$

where,

fvt* = in-plane shear stress in flange plate due to torsion

fvj* = shear stress at the junction of the web and the flange

fvf = nominal yield stress of the material of flange plate

2.6 Strength of Longitudinal Flange Stiffeners

AS5100.6: 2017
Cl 7.3.3.3

The compressive flange plate shall be checked for the following:

a) $f_s^* + 2.5 f_{vt}^* k_{s1} < \phi k_{L1} f_{ys}$ 2-6

b) $f_s^* + 2.5 f_{vt}^* k_{s2} < \phi k_{L2} f_{ye}$

where,

f_s^* = longitudinal stress at the centroid of the effective section of the stiffener (compression taken as positive)

f_{vt}^* = in-plane shear stress in the flange plate due to torsion (always positive)

ks1, ks2 = coefficients used for the design of longitudinal flange stiffeners obtained from Figure 7.3.3.3 of AS5100.6:2017

kL1, kL2 = reduction factors obtained from figure 7.3.3.3 of AS5100.6:2017

fys = yield stress of the stiffener material

$$f_{ye} = \sqrt{f_{yf}^2 - 3\left(\frac{f_{vf}^*}{\phi}\right)^2}$$

fyf = nominal yield stress of flange plate material

$f_{vf}^* = f_{vt}^* + (0.5\, f_{vj}^*)$

f_{vj}^* = shear stress in flange plate at the junction with the web of beam due to shear force on beam

2.7 Shear Capacity

AS5100.6:2017
Cl 5.10.1

The shear capacity of the web (Vv) shall satisfy the following conditions:

$$V^* \leq \phi V_V$$

where,

V* = design shear force from the analysis

ϕ = capacity reduction factor specified in section 1.7.2

V_v = nominal shear capacity of the web

AS5100.6:2017
Cl 5.10.1

The nominal shear capacity of the web shall be determined considering the shear stress distribution is approximately uniform.

Depending upon the ratio of the depth of the deepest web panel (d_p) to the thickness of the web (t_w), the nominal shear yield capacity of the web or the nominal buckling yield capacity of the web is determined. Refer to the table given below:

[Table 2.1] Shear capacity of a section

	Shear capacity shall be in accordance with
$\frac{d_p}{t_w} \leq \frac{82}{\sqrt{\frac{f_y}{250}}}$	$V_v = V_w$
$\frac{d_p}{t_w} > \frac{82}{\sqrt{\frac{f_y}{250}}}$	$V_v = V_b$

2.7.1 Shear Yield Capacity

AS5100.6:2017
Cl 5.10.4

The nominal shear yield capacity of the considered section shall be determined in accordance with the equation given below:

$$V_w = 0.6 f_y A_w$$

where,

Aw = gross cross-section area of the web of the considered section

2.7.2 Shear Buckling Capacity

The shear buckling capacity of the section shall be calculated depending on whether the web is unstiffened or stiffened.

a) Unstiffened web

If the transverse stiffeners are not c 2-7 section, then the program will consider the web to be unstiffened and the nominal shear buckling capacity shall be calculated as:

AS5100.6:2017 Cl 5.10.5.1

$$V_b = \alpha_v V_w \leq V_w$$

where,

αv = shear buckling coefficient of the web

$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\right)\sqrt{\frac{f_y}{250}}}\right]^2$$

b) Stiffened web

If the transverse stiffeners are defined for the section, then the program will consider the web to be stiffened and the nominal shear buckling capacity of the stiffened web with s/dp ≤ 3.0, shall be calculated as:

AS5100.6:2017 Cl 5.10.5.2

$$V_b = \alpha_v \alpha_d \alpha_f V_w \leq V_w$$

where,

$$\alpha_v = \left[\frac{82}{\left(\frac{dp}{t_w}\right)\sqrt{\frac{f_y}{250}}}\right]^2 \left[\frac{0.75}{\left(\frac{s}{d_p}\right)^2} + 1\right] \leq 1.0 \text{ if } 1.0 \leq \frac{s}{d_p} \leq 3.0$$

$$\alpha_v = \left[\frac{82}{\left(\frac{dp}{t_w}\right)\sqrt{\frac{f_y}{250}}}\right]^2 \left[\frac{1.0}{\left(\frac{s}{d_p}\right)^2} + 0.75\right] \leq 1.0 \text{ if } \frac{s}{d_p} \leq 1.0$$

α_d = tension field coefficient for web shear buckling

$$= 1 + \frac{1 - \alpha_v}{1.15\alpha_v\sqrt{1 + \left(\frac{s}{d_p}\right)^2}}$$

For s/dp > 3.0, the value of product $\alpha_v\alpha_d$ is taken from table 5.10.5.2 of AS5100.6:2017.

However, if s/dp > 3.0 and $\left(\frac{d_p}{t_w}\right)\sqrt{\frac{f_y}{250}}$ < 90 or >180, value of $\alpha_v\alpha_d$ does not exist.

Hence, $\alpha_v\alpha_d$ = 0 will be considered.

The flange restraint factor αf shall be taken as:

i) αf = 1.0; or

ii) $\alpha_f = 1.6 - \frac{0.6}{\sqrt{\left[1 + \left(\frac{40 b_{fo} t_f^2}{d_1^2 t_w}\right)\right]}}$;

for the webs in which longitudinal stiffeners are not provided.

The b_{fo} shall be taken as the least of the following:

i) $\frac{12 t_f}{\sqrt{\frac{f_y}{250}}}$;

ii) The distance from the mid-plane of the web to the nearer edge of the flange.

iii) Half the clear distance between the webs if there are two or more webs.

2.8 Interaction of Shear and Bending

A composite beam subjected to combined bending and shear shall satisfy:

AS5100.6:2017
Cl 5.11.3

$$V^* \leq \phi V_{vm}$$

where,

$V^* = V_v$ for $M^* \leq 0.75\phi M_s$

$= V_v\left[2.2 - \left(\frac{1.6M^*}{\phi M_s}\right)\right]$ for $.75\phi M_s \leq M^* \leq \phi M_s$

V_v = nominal shear capacity of web in shear alone

M_s = nominal section moment capacity

2.9 Shear Capacity of Web with Longitudinal Stiffeners

For beams with the longitudinal web stiffeners, the yielding, and the buckling of each of the web panels shall be considered.

2.9.1 Yielding of Web Panel

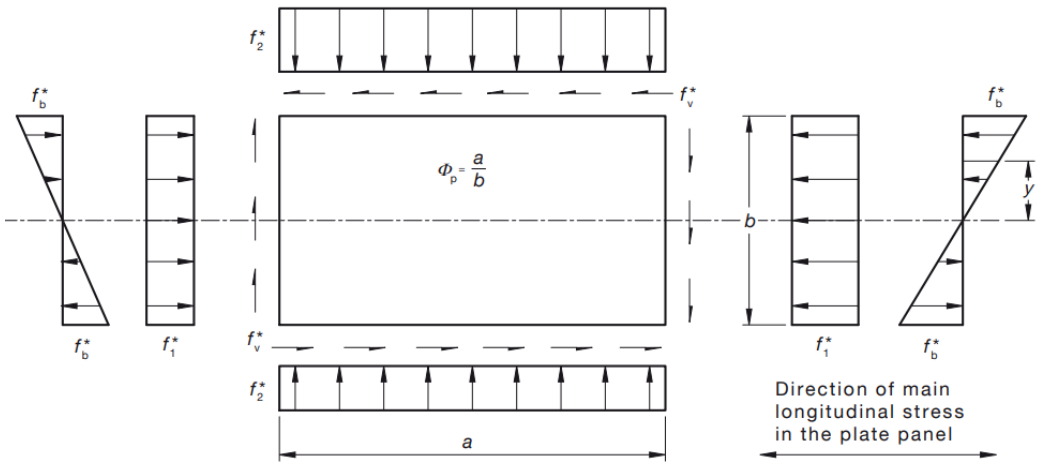
To consider the yielding of web with longitudinal stiffeners, the condition shown below must be satisfied:

AS5100.6:2017
Cl 7.4.2

$$\sqrt{\left(f_{le}^*\right)^2 + (f_2^*)^2 - f_{le}^* f_2^* + 3(f_v^*)^2} \leq \phi f_y$$

where,

$f_1^*, f_2^*, f_b^*, f_v^*$ = coexistent components of the stress as shown in the figure below



[Fig. 2.5] Stress on Web Panels

f_1^* = mean longitudinal in-plane stress on a cross-section of the panel (taken as positive in compression)

f_2^*= in-plane transverse stress (assumed as zero, as it cannot be considered through 1D analysis)

f_b^* = maximum longitudinal stress due to in-plane bending of individual panel (taken as positive in compression)

f_v^* = average design shear stress due to applied shear force

f_{le}^* = effective longitudinal in-plane stress in the plate panel, which shall be considered using the following conditions.

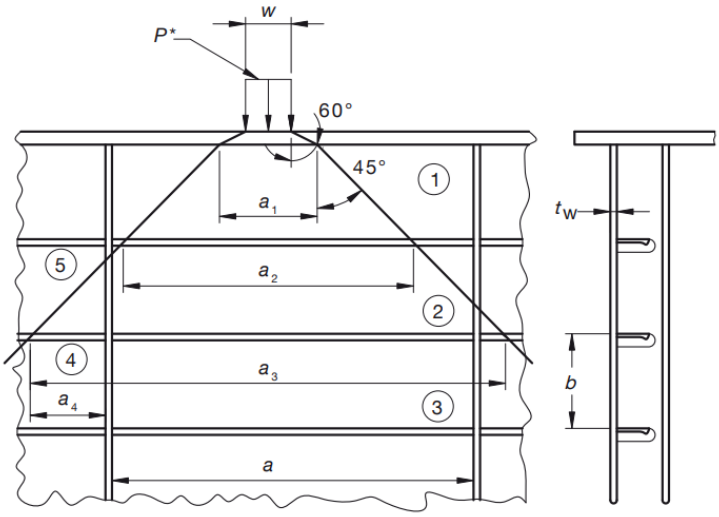
i) When transverse stress f_2^* is absent
$f_{le}^* = f_1^* + 0.77f_b^*$

ii) When transverse stress f_2^* is present.
$f_{le}^* = f_1^* + kf_b^*$
where,
k = 2y/b or 0.77, whichever is smaller

2.9.2 Buckling of Web Panel

The transverse stress shall be calculated at each of the panel locations as per the dispersion criteria specified below:

AS5100.6:2017
Cl 7.4.3.1



(a) Typical panels

Panel	Transverse stress f_2^* to be used for each panel over length a
1	$\frac{P^*}{a_1 t_w}$
2	$\frac{P^*}{a_2 t_w}$
3	$\frac{P^*}{a_3 t_w}$
4	$\frac{P^*}{a_3 t_w}$
5	0

(b) Treatment of panels

[Fig. 2.6] Load Dispersion through the Longitudinally Stiffened Web

2.9.2.1 Restraint for Web Panels

Whether the web panel is restrained or unrestrained shall be decided based on the following formulations. It could be noted that all the interior web panels i.e. not adjacent to a flange, may be deemed to be restrained.

AS5100.6:2017
Cl 7.4.3.2

The web panels adjacent to the flanges are deemed restrained if:

a) λ<24; or

b) i) $m_{fw} > 0.00025(\lambda - 24)\left[\frac{f_{yf}^2}{f_{yf}^2 - \left(\frac{f_f^*}{\phi}\right)^2}\right]$ for 24 ≤λ ≤84, but λ be taken as 84 for λ>84

if

ii) $\lambda > \left[66 + \left(\frac{28}{(\phi_p)^2}\right)\right]$ then mfw is greater than the limiting value as per figure 7.4.3.2.2 of AS5100.6:2017.

where,

$$\lambda = \frac{b}{t_w}\sqrt{\frac{f_{yw}}{355}}$$

$$m_{fw} = \frac{f_{yf} b_{fe} t_f^2}{2 f_{yw} b^2 t_w}$$

a,b = length and width of the panel

tw = thickness of the web

fyw = nominal yield stress of the web material

fyf = nominal yield stress of the flange material

bfe = effective width of the flange, taken as smallest of:

A. $10 t_f \sqrt{\frac{355}{f_{yf}}}$

B. distance from the mid-plane of the web to the nearer edge of the flange, i.e., taken as zero if there is no flange outstand or

C. if there are two or more webs, half the clear distance between webs.

where,

tf = thickness of the flange

fvf = nominal yield stress of the flange material

ϕp = aspect ratio a/b as shown on Figure 7.4.2 of AS5100.6:2017
ff*= longitudinal stress in the flange plate, including any redistribution of stresses

2.9.2.2 Buckling Coefficients

The buckling coefficients mentioned below shall be obtained from the AS5100.6:2017 clause 7.4.3.3.

a) Axial coefficient (K1)
b) Shear coefficient (Kv)
c) Bending Coefficient (Kb)
d) Transverse coefficient (K2)

2.9.2.3 Interaction Buckling Criterion

For each panel of the section the following condition shall be satisfied:

AS5100.6:2017 Cl 7.4.3.4

$$m_c + m_b + 3m_v \leq 1$$

The check is based on whether or not the transverse stress is present in the panel.

a) Transverse stress is absent in the panel.

$$m_c = \frac{f_1^*}{\phi f_{yw} K_1 (1-\rho)}$$

$$m_b = \left[\frac{f_b^*}{\phi f_{yw} K_b (1-\rho)}\right]^2$$

$$m_v = \left(\frac{f_v^*}{\phi f_{yw} K_v}\right)^2$$

where,
f1*= as described in 2.9.1
fyw = nominal yield stress of the web material
K1, Kb, Kv = as described in 2.9.2.2
fb*, fv* = as described in 2.9.1

b) Transverse stress is present in the panel.

$m_c = \sqrt{(m_1 + m_2)}$ for positive value of (m1+m2)

$m_c = -\left\{\sqrt{-(m_1 + m_2)}\right\}$ for negative value of (m1+m2)

$m_1 = \left[\frac{f_1^*}{\phi f_{yw} K_1 (1-\rho)}\right]^2$ shall be taken as negative if f1*is tensile

$m_v = \left(\frac{f_2^*}{\phi f_{yw} K_2}\right)^2$ shall be taken as negative if f2*is tensile

where,
m1, m2 = factors for checking the plate panel interaction buckling criterion

2.9.3 Strength of Longitudinal Web Stiffeners

AS5100.6:2017 Cl 7.4.4.2

The strength of the longitudinal stiffeners shall be determined by using the following equation:

$$f_{se}^* \leq \phi f_{Ls}$$

where,
f_{se}^* = equivalent axial stress in the stiffener for buckling
f_{Ls}= limiting stiffener stress obtained from the Figure 7.4.4.2 of AS5100.6:2017
λ = slenderness ratio

$$= \frac{a}{r_{se}} \sqrt{\frac{f_{ys}}{355}}$$

$$f_{se}^* = k_s' f_1^* + \left(2.5 f_v^* + \left(\frac{a^2}{b^2}\right) f_2^*\right) \frac{b t_w k_s}{A_{se}}$$

k_s' = coefficient used in determining the effective stiffener stress
= 1.0 for continuous longitudinal stiffeners
= 2.5ks for discontinuous longitudinal stiffeners

f_1^*= longitudinal stress along the stiffener connection centre-line derived without redistribution, taken as positive if compressive
f_v^*= average shear stress
a = clear distance between transverse web stiffeners
b = mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration
f_2^* = coexistent transverse stress, if any, taken as positive if compressive
tw = web plate thickness
ks = coefficient obtained from Figure 7.4.4.2 of AS5100.6:2017 using the value of λ
rse = radius of gyration of the effective stiffener section about an x-x axis parallel to the web
Ase = area of effective stiffener section

2.10 Verification of Design/ Check Results

Midas Civil provides the design output in various formats. The following design outputs are available for the user:

i) Design Result Tables
ii) Design Report (Microsoft Word .docx file)
iii) Steel Composite Design Results Diagram

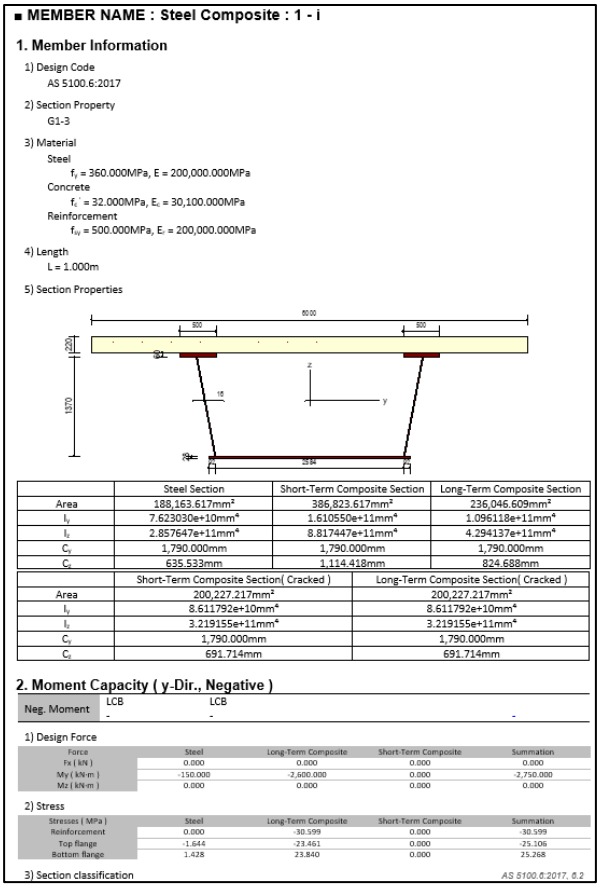
The design results shall be checked as follows.

- *Design > Composite Design > Design Results Tables*

	Elem	part	Lcom	Type	CHK	Demand (kN/m²)	ΦkL*fy (kN/m²)
▶	1	I[1]	scLCB9	-	OK	37171.010	214791.3424
	1	J[2]	scLCB9	-	OK	10069.338	224719.1486
	2	I[2]	scLCB1	-	OK	0.0000	229936.7539
	2	J[3]	scLCB1	-	OK	0.0000	229936.7539

[Fig. 2.7] Result Table

Design > Composite Design > Print Result



■ **MEMBER NAME : Steel Composite : 1 - i**

1. Member Information

1) Design Code
AS 5100.6:2017

2) Section Property
G1-3

3) Material
Steel
f_y = 360.000MPa, E = 200,000.000MPa
Concrete
f_c' = 32.000MPa, E_c = 30,100.000MPa
Reinforcement
f_{sy} = 500.000MPa, E_s = 200,000.000MPa

4) Length
L = 1.000m

5) Section Properties

	Steel Section	Short-Term Composite Section	Long-Term Composite Section
Area	188,163.617mm²	386,823.617mm²	236,046.609mm²
I_y	7.623030e+10mm⁴	1.610550e+11mm⁴	1.096118e+11mm⁴
I_z	2.857647e+11mm⁴	8.817447e+11mm⁴	4.294137e+11mm⁴
C_y	1,790.000mm	1,790.000mm	1,790.000mm
C_z	635.533mm	1,114.418mm	824.688mm

	Short-Term Composite Section(Cracked)	Long-Term Composite Section(Cracked)
Area	200,227.217mm²	200,227.217mm²
I_y	8.611792e+10mm⁴	8.611792e+10mm⁴
I_z	3.219155e+11mm⁴	3.219155e+11mm⁴
C_y	1,790.000mm	1,790.000mm
C_z	691.714mm	691.714mm

2. Moment Capacity (y-Dir., Negative)

Neg. Moment	LCB	LCB	
	-	-	-

1) Design Force

Force	Steel	Long-Term Composite	Short-Term Composite	Summation
Fx (kN)	0.000	0.000	0.000	0.000
My (kN·m)	-150.000	-2,600.000	0.000	-2,750.000
Mz (kN·m)	0.000	0.000	0.000	0.000

2) Stress

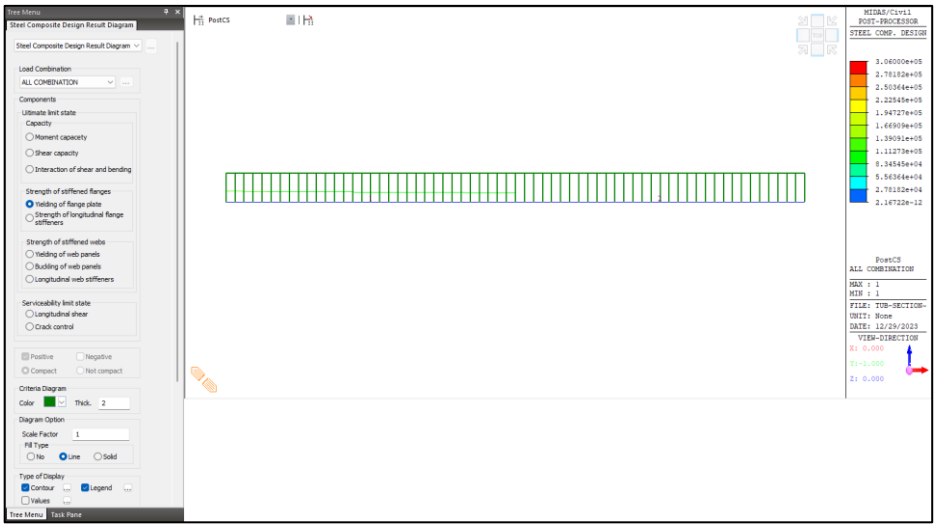
Stresses (MPa)	Steel	Long-Term Composite	Short-Term Composite	Summation
Reinforcement	0.000	-30.599	0.000	-30.599
Top flange	-1.644	-23.461	0.000	-25.106
Bottom flange	1.428	23.840	0.000	25.268

3) Section classification

AS 5100.6:2017, 6.2

[Fig. 2.8] Design Report

Design > Composite Design > Steel Composite Design Result Diagram



[Fig. 2.9] Result Diagram

Serviceability Limit State

The serviceability limit state shall be checked for the crack control of the slab and the longitudinal shear force in the shear connectors.

3.1 Crack Control of Slab

AS5100.5:2017 Cl 9.4.1

For the reinforced slabs, the crack shall be deemed as controlled if all the following requirements are satisfied:

a) The reinforcement provided in the slab shall be greater than the minimum reinforcement required. The minimum reinforcement shall be calculated using the following formula:

AS5100.5:2017 Cl 9.1.1

$$A_{st} = 0.0025bd$$

where,
b = width of the slab
d = depth of the slab

b) The centre-to-centre spacing of the bars (s) shall not exceed the lesser of
   i) $2D_s$
   ii) 300mm

c) For SLS combinations, the maximum tensile stress in the steel bars (f_s^*) shall not be greater than the maximum stresses given as:
   i) For the largest nominal diameter (d_b) of the bar in the tensile zone, the maximum tensile stress is as shown in the table below:

[Table 3.1] Maximum Steel Stress for Flexure in Reinforced Slab - Nominal Diameter

AS5100.5:2017 Table 9.4.1(A)

Nominal bar diameter (d_b) in mm	Maximum steel stress ($f_{scr,D}$) in MPa	
	For D_s≤300mm	For D_s>300mm
8	345	400
10	320	360
12	300	330
16	265	280
20	240	
24	210	

   ii) For maximum centre-to-centre spacing between the bars in the tensile zone, the maximum stress is as shown in the table below:

[Table 3.2] Maximum Steel Stress for Flexure in Reinforced Slab - Centre-to-Centre Spacing

AS5100.5:2017 Table 9.4.1(B)

Centre-to-centre spacing in mm	Maximum steel stress ($f_{scr,S}$) in MPa
50	360
100	320
150	280
200	240
250	200
300	160

Crack Control of Slab

The results corresponding to crack control in slabs shall be viewed in the form of a result table or the design report.

Design > Composite Design > Design Result Tables/Print Result …

8. Crack Control of Slab

Stress	LCB	scLCB11	
	f_s / f_{scr}	28.652MPa / 265.000MPa = 0.108	OK

1) Design Force

Force	Steel	Long-Term Composite	Short-Term Composite	Summation
Fx (kN)	0.000	0.000	0.000	0.000
My (kN-m)	-150.000	-2,600.000	0.000	-2,750.000
Mz (kN-m)	0.000	0.000	0.000	0.000

2) Effective Section Property

	Steel Section	Short-Term Composite Section	Long-Term Composite Section
Area	175,536.701mm²	187,600.301mm²	187,600.301mm²
I_y	7.119079e+10mm⁴	8.014047e+10mm⁴	8.014047e+10mm⁴
C_z	677.583mm	734.841mm	734.841mm

3) Rebar

$f_s^* = \frac{M_y}{I_{y,eff}} z = -28.652MPa \le f_{scr} = 265.000MPa$

where,
$I_{y,eff}$ = 8.014047e+10mm⁴
z = 883.159mm
$f_{scr,D}$ = 265.000MPa (Nominal Diameter)
$f_{scr,S}$ = 240.000MPa (Center-to-Center Spacing)
f_{scr} = max($f_{scr,D}$, $f_{scr,S}$) = 265.000MPa

- Check minimum area of reinforcement
A_s = 6,031.800mm² > A_{st} = 2,400.000mm²
where,
A_{st} = 0.0025bd = 2,400.000mm²
b = 6,000.000mm
d = 160.000mm

- Check the maximum spacing of rebars
s = 200.000mm ≤ s_{max} = 300.000mm
where,
D_s = 220.000mm
S_{max} = min($2D_s$, 300) = 300.000mm

[Fig. 3.1] Result output for crack control of slabs

3.2 Longitudinal Shear

AS5100.6:2017 Cl 4.8

The longitudinal shear check shall be carried out for a section under consideration depending upon the shear connectors' data provided for design.

The shear force per unit length for a section shall be calculated as follows:

$$v_L^* = \frac{V^* A_t y_c}{I_t}$$

where,
V^* = design shear force for the section
A_t = area of the section to one side of the shear plane
y_c = distance of the neutral axis of the transformed cross-section to the centroid of the area A_t
I_t = second moment of inertia of the transformed composite cross-section (short-term composite section)

AS5100.6:2017 Cl 6.8.3.2

The following check shall be performed for the serviceability load combinations under consideration:

$$v_L^* \le \phi v_{Ls}$$

v_{Ls} = permissible longitudinal shear force per unit length at serviceability state

$$v_{Ls} = 0.55 n f_{ks}$$

where,
ϕ = capacity reduction factor
n = number of shear connectors per unit length of the section
f_{ks} = characteristic shear capacity of the shear connectors, which shall be considered as the minimum of the following:

AS5100.6:2017 Table 3.3

AS5100.6:2017 Cl 6.8.4.4

$$f_{ks} = min(\ \alpha_{shear}\ d_{bs}^2 f_{uc}, 0.29 d_{bs}^2 \sqrt{f'_{cy} E_c}\ )$$

where,

d_{bs} = nominal shank diameter of a shear stud in the range [15.9,25] mm

f_{uc} = ultimate tensile strength of the shear connector material limited to 500 MPa

f_{cy}' = characteristic strength of the concrete at the age being considered, but not greater than f_c' or else f_{cy}' = 100 MPa

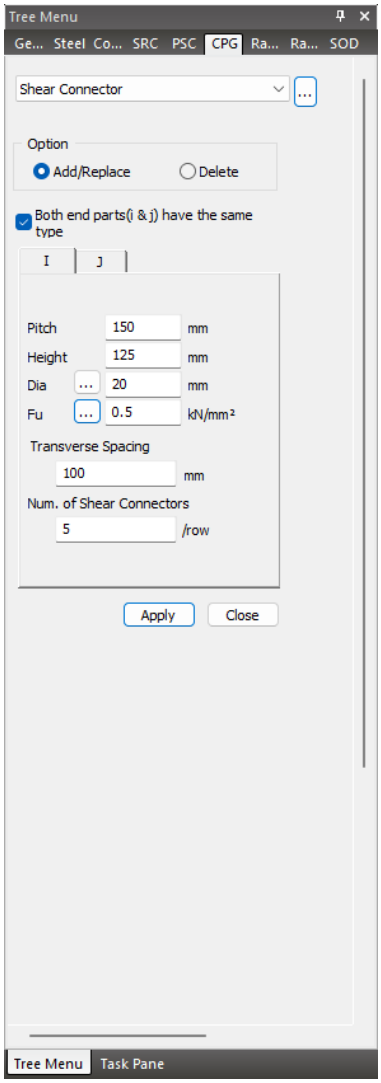
E_c = elastic modulus of slab concrete corresponding to the relevant value of f_{cmi}

α_{shear} = 0.70 for headed stud connectors welded per Clause 2.6 of AS5100.6-2017

Shear Connectors

The data relating to shear connectors, such as the diameter, height, grade of steel, pitch, spacing, etc., can be specified in the "Shear Connectors" dialog box. The built-in database for the diameter of the shear connectors and their grade can be specified by clicking on the button provided in front of each mentioned input field.

Design > Composite Design > Shear Connector



[Fig. 3.2] Shear connector data for design

The data provided can be checked and modified by clicking on the button at the top of the tree menu.

	Elem	Both-I,J	Pitch-I (mm)	Height-I (mm)	Dia-I (mm)	Fu-I (kN/mm²)	Spacing Shear Conn.-I (mm)	Num. of Shear Conn.-I
	1	☑	150.00	125.00	20.00	0.50	100.00	5
	2	☑	150.00	125.00	20.00	0.50	100.00	5

[Fig. 3.3] Input data for the design of shear connector

❒ Longitudinal Shear Check

The results corresponding to longitudinal shear shall be viewed in the form of a result table or the design report.

- *Design > Composite design > Design Result Tables/Print Result …*

5. Longitudinal Shear for SLS

Shear Force	LCB	SLS	
	$v_L / \phi v_{Ls}$	3,834.248kN/m / 4,209.302kN/m = 0.911	OK

1) Design longitudinal shear force per unit length — *AS 5100.6:2017, 4.8*

$$v_L^* = \frac{V^* A_t z_c}{I_t} = 3{,}834.248\text{kN/m}$$

Where

$V^* = 6{,}400.915\text{kN}$

$A_t = 103{,}730.000\text{mm}^2$

$z_c = 332.410\text{mm}$

$I_t = 5.756252\text{e+}10\text{mm}^4$

2) Characteristic shear capacity of shear connectors — *AS 5100.6:2017, 6.8.4.4*

$f_{ks} = \alpha_{shear}\, d_{bs}^2 f_{uc} = 201.600\text{kN}$

$f_{ks} = 0.29 d_{bs}^2 \sqrt{f'_{cy} E_c} = 191.332\text{kN}$

$f_{ks} = 191.332\text{kN}$

Where

$d_{bs} = 24.000\text{mm}$

$f_{uc} = 500.000\text{MPa}$

$f'_{cy} = 40.000\text{MPa}$

$\alpha_{shear} = 0.700$

3) Design for longitudinal shear — *AS 5100.6:2017, 6.8.3.2*

$\phi v_{Ls} = 4{,}209.302\text{kN/m}$

$v_{Ls} = 0.55 n f_{ks} = 4{,}209.302\text{kN/m}$

$n = 4 / 100.000\text{mm}$

$\phi = 1.000$

[Fig. 3.4] Design Result Output for Longitudinal Shear