Notation

- A_c : Area of concrete slab within effective width (= $D_s B_{eff}$), mm^2
- A_s : Area of steel section, mm^2
- A_{sa} : Cross-sectional area of steel headed stud anchor, mm^2
- A_{sr} : Area of developed longitudinal reinforcing steel within the effective width of the concrete slab, mm^2
- A_w : Area of web, the overall depth times the web thickness, mm^2
- B : Width of steel section; distance between outer faces of webs, mm
- B_{ay} : Distance between adjacent beams, mm
- B_{eff} : Effect width of slab, mm
- C : Compression force in the concrete slab, kN
- C_b : The lateral-torsional buckling modification factor, dimensionless
- C_f : Compression force in concrete slab for fully composite beam, kN
- C_v : The web shear strength coefficient, dimensionless
- C_y : Distance from the centroid of steel section to the outer surface of bottom flange, mm
- $C_{y,c}$: Distance from the centroid of composite section to the lower surface of bottom flange, mm
- C_{yp} : Distance from the center of mass to the outer surface of bottom flange, mm
- D_s : Depth of slab, mm
- D_{tr} : Distance from the centroid of top rebars to the top of the slab, mm
- E : Elastic modulus of the material, MPa
- E_c : Elastic modulus of concrete, MPa
- E_r : Elastic modulus of reinforcing bar, MPa
- E_s : Elastic modulus of steel, MPa
- F_{cr} : Lateral-torsional buckling stress for the section as determined by analysis, MPa
- $F_{cr,bp}$: Local buckling strength of bottom plate, MPa
- F_L : Nominal compressive strength above which the inelastic buckling limit states apply, MPa
- F_u : Specified minimum tensile strength of a steel headed stud anchor, MPa
- F_y : Yield strength of structural steel, MPa
- F_{yr}, f_{yr} : Yield strength of the slab reinforcements, MPa
- H : Overall height of steel (U-shaped) section, mm
- I_{equiv} : The equivalent moment of inertia of composite section, mm^4
- I_{tr} : Moment of inertia for the uncracked fully composite section , mm^4
- I_s : Moment of inertia for the structural section, mm^4
- I_x : Moment of inertia of steel section about the x-axis, mm^4
- I_y : Moment of inertial of steel section about the y-axis, mm^4
- I_{yc} : Moment of inertial of the compression flange about the y-axis, mm^4
- $I_{y,bf}$: Moment of inertia of the bottom flange about y-axis, mm^4
- $I_{y,tf}$: Moment of inertia of the top flanges about y-axis, mm^4
- $I_{y,w}$: Moment of inertia of the webs about the y-axis, mm^4

Notation

- J : Torsional constant, mm^4
- L : Length of beam, mm
- L_b : Length between points that are either braced against lateral of the compression flange or braced against twice of the cross section.
- L_p : The limiting laterally unbraced length for the limit state of yielding, mm
- L_r : The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, mm
- M_A : Maximum absolute moment value at quarter point of the unbraced segment, kN-m
- M_B : Maximum absolute moment value at centerline of the unbraced segment, kN-m
- M_C : Maximum absolute moment value at three-quarter point of the unbraced segment, kN-m
- M_{cr} : Moment resistance corresponding to compression flange local buckling, kN-m
- M_{max} : Maximum absolute moment value in the unbraced segment, kN-m
- M_n : Nominal flexural strength, kN-m
- $M_{n,neg}$: Nominal flexural strength in negative bending, kN-m
- $M_{n,pos}$: Nominal flexural strength in positive bending, kN-m
- $M_{nc,pos1}$: Nominal moment for the compression flange yielding under positive bending, kN-m
- $M_{nc,pos3}$: Nominal moment for the tension flange yielding under positive bending, kN-m
- $M_{nc,pos4}$: Nominal moment for the lateral-torsional buckling under positive bending, kN-m
- $M_{nc,neg1}$: Nominal moment for the compression flange yielding under negative bending, kN-m
- $M_{nc,neg2}$: Nominal moment for the compression flange local buckling under negative bending, kN-m
- $M_{nc,neg3}$: Nominal moment for the tension flange yielding under negative bending, kN-m
- $M_{nc,neg4}$: Nominal moment for the lateral-torsional buckling under megative bending, kN-m
- M_p : plastic moment strength (= F_yZ_x), mm^4
- $M_{u,neg}$: Maximum required negative flexural strength, kN-m
- $M_{u,pos}$: Maximum positive flexural strength, kN-m
- M_y : Yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kN-m
- M_{yc} : Yield moment in the compression flange, kN-m
- M_{yt} : Yield moment in the tensile flange, kN-m
- P_{bc} : Compressive strength of the bottom concrete, kN
- P_{bp} : Yield strength of the bottom flange, kN
- P_{bw} : Yield strength of the web in tension, kN
- P_{sh} : Shrinkage load, kN
- P_{slab} : Compressive force of the concrete slab, kN
- P_{sr} : Tensile yielding of the slab reinforcement, kN
- P_{steel} : Tensile yielding strength of the steel section, kN
- P_{tf} : Yield strength of the top flange, kN
- P_{tw} : Yield strength of the web in compression, kN

Notation

- P_w : Yield strength of the web, kN
- Q_n : Shear strength of the steel headed stud or steel channel anchor, kN
- R_g : Coefficient to account for group effect of shear studs, dimensionless
- R_m : Modifier for singly symmetric sections, dimensionless
- R_p : Position effect factor for shear studs, dimensionless
- R_{pc} : Web plastification factor corresponding to the compressive flange yielding limit state, dimensionless
- R_{pt} : Web plastification factor corresponding to the tension flange yielding limit state, dimensionless
- S_x : Elastic section modulus about the x-axis, mm^3
- $S_{x,bf}$: Elastic section modulus referred to bottom flange ($= I_x / C_y$), mm^3
- S_{xc} : Elastic section modulus referred to the compressive flange, mm^3
- S_{xt} : Elastic section modulus referred to the tensile flange [$=I_x / (H - C_y)$], mm^3
- $S_{x,tf}$: Elastic section modulus referred to top flange [$=I_x / (H - C_y)$], mm^3
- T : Tensile force in beam section, kN
- V_n : Nominal shear strength, kN
- $V_{q,neg}$: Shear force between the steel beam and the concrete slab transferred by steel anchors, in the region of beam carrying negative moment, kN
- $V_{q,pos}$: Shear force between the steel beam and the concrete slab transferred by steel anchors, in the region of beam carrying positive moment, kN
- V_{stud} : Shear strength of steel headed studs, kN
- V_u : Maximum required shear strength, kN
- V' : The nominal shear force between the steel beam and the concrete slab transferred by steel anchors, kN
- W_d : Self-weight of the TSC beam, kN/m
- W_f : Superimposed load, kN/m^2
- W_l : Live load, kN/m^2
- W_s : Self-weight of the concrete slab, kN/m^2
- Z_x : Plastic section modulus of steel section, mm^3
- a : Depth of the compression block in the concrete slab, mm
- a_w : Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components, dimensionless
- b_f: Width of the top flange, mm
- b_{fc} : Width of the compression flange, mm
- $b_{f,in}$: Width of inner top flange, mm
- $b_{f,out}$: Width of outer top flange, mm
- b_p : Width of the bottom flange, mm
- d_{bp} : Distance from P_{bp} to NA, mm
- d_{bw} : Distance from P_{bw} to NA, mm
- d_{co} : Distance between the slab surface and the center of the longitudinal slab reinforcements, mm
- d_{na} : Distance from NA to the top of the slab, mm
- d_{rb} : Distance from T to NA, mm
- d_s : Distance from the centroid of the compression force C, in the concrete to NA ($= d_{pna} - 0.5a$), mm

Notation

- d_{tf}: Distance from P_{tf} to NA, mm
- d_{tw} : Distance from P_{tw} to NA, mm
- d_w : Distance from P_w to NA, mm
- f_{ck} : Compressive strength of concrete, MPa
- f'_c : Specified compressive strength of concrete, MPa
- h : height of the web plate, mm
- h_o : Distance between flange centroids, mm
- h_s : Height of the stud, mm
- k_c : Coefficient for slender unstiffened elements, dimensionless
- k_v : Coefficient of plate shear buckling, dimensionless
- l_{neg} : Negative moment region
- l_{pos} : Positive moment region
- $n_{s,neg}$: The number of shear studs in the region of beam carrying negative moment, dimensionless (EA)
- $n_{s,pos}$: The number of shear studs in the region of beam carrying positive moment, dimensionless (EA)
- r_t : Effective radius of gyration for lateral-torsional buckling, mm
- t_f: Thickness of the top flange, mm
- t_p : Thickness of the bottom flange, mm
- t_w : Thickness of the web, mm
- w_c : Weight of concrete per unit volume, kN/m^3
- δ_{add} : Additional deflection after concrete curing, mm
- δ_{comst} : Short-term deflection of the composite TSC beam (= δ_{const} + δ_{add}), mm
- δ_{const} : Deflection caused by load combination considered in construction stage, mm
- ϕ_b : The reduction factor for the design flexural strength (= 0.90), dimensionless
- ϕ_v : The reduction factor for the design shear strength (= 0.90), dimensionless
- λ : Width-to-thickness ratio of the element, dimensionless
- λ_{bp} : Width-to-thickness ratio of bottom plate, dimensionless
- λ_{pf}: The limiting slenderness parameter for compact flange, dimensionless
- λ_{pw} : The limiting slenderness parameter for compact web, dimensionless
- λ_{rf}: The limiting slenderness parameter for non-compact flange, dimensionless
- λ_{rw} : The limiting slenderness parameter for non-compact web, dimensionless
- λ_{tf}: Width-to-thickness ratio of top flange, dimensionless
- λ_w : Width-to-thickness ratio of web, dimensionless

2. Design Procedure

Design guide of the TSC beam according to AISC 360-16

Design	Category	Chapter	Provision
General provisions	U-shaped steel section	Chapter B.	B.4.1. Classification of sections for local buckling
	Composite section	Chapter I.	I.1.3. Material limitations I.1.4 Classification of **filled composite sections** for local buckling
Flexure	U-shaped steel section	Chapter F.	F1. General provisions F4. Other I-shaped members with compact or noncompact webs bent about their major axis F5. Doubly and symmetric and singly symmetric I-shaped members with slender webs about their major axis
	Composite section	Chapter I.	I3.1 General I.3.2c Composite beams with formed steel deck I.3.2d **Load transfer between steel beam and concrete slab** I.3.4 **Filled composite members** > 4b. Flexural strength
Shear	U-shaped steel section	Chapter G.	G1. General provisions G4. I-shaped members and channels
	Composite section	Chapter I.	I4.1 Filled and encased composite members
Deflection	U-shaped steel section	Chapter I.	I3.1b Strength during construction *Comment I3.2*
	Composite section		

2. Design Procedure

Section Properties of the TSC beam

- U-shaped steel section without concrete infill

  - Area of steel section $A_s = 2b_f t_f + 2ht_w + b_p t_p$, mm^2

  - Distance from the centroid of steel section to the outer surface of bottom flange, mm

$$C_y = \frac{2b_f t_f \times \left(H - t_f/2\right) + 2ht_w \times \left(h/2\right) + b_p t_p \times \left(t_p/2 + c_o\right)}{A_s}$$

  - Moment of inertia of steel section about the x-axis, mm^4

$$I_x = 2\left\{\frac{b_f}{12}t_f^{\,3} + b_f t_f\left(H - C_y - t_f/2\right)^2\right\} + 2\left\{\frac{t_w}{12}h^3 + t_w h\left(h/2 - C_y\right)^2\right\}$$
$$+\left\{\frac{b_p}{12}t_p^{\,3} + b_p t_p\left(C_y - t_p/2 - c_o\right)^2\right\}$$

  - Elastic section modulus referred to bottom flange $S_{x,bf} = I_x/(C_y - c_o)$, mm^3

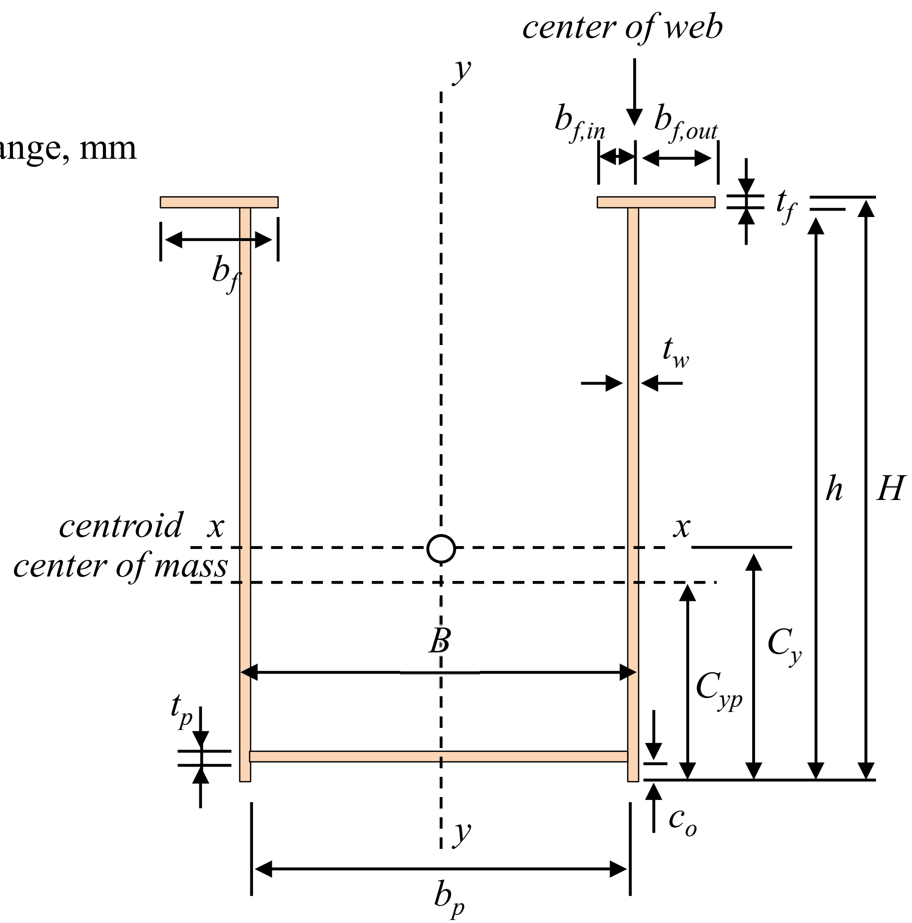
  - Elastic section modulus referred to top flange $S_{x,tf} = I_x/(H - C_y)$, mm^3

  - Distance from the center of mass to the outer surface of bottom flange, mm

$$C_{yp} = \frac{\left(A_s/2 - t_p b_p\right)}{2t_w}$$

  - Plastic section modulus

$$Z_x = 2b_f t_f \times \left(H - C_{yp} - t_f/2\right) + 2\left(h - C_{yp}\right)t_w \times \left(h - C_{yp}\right)/2 + 2t_w C_{yp} \times C_{yp}/2 + b_p t_p \times \left(C_{yp} - t_p/2 - c_o\right)$$



2. Design Procedure

Section Properties of the TSC beam

- U-shaped steel section without concrete infill

  - Moment of inertia of the top flange about the y-axis $I_{y,tf} = 2\left\{\frac{t_f}{12}b_f^{\,3} + b_f t_f \left(B/2 + t_w/2 + b_{f,out} - b_f/2\right)^2\right\}$

  - Moment of inertia of the web about the y-axis $I_{y,w} = 2\left\{\frac{h}{12}t_w^{\,3} + h t_w \left(B/2 - t_w/2\right)^2\right\}$

  - Moment of inertia of the bottom flange about the y-axis $I_{y,bf} = \frac{t_p}{12}b_p^{\,3}$ - Moment of inertia about the y-axis $I_y = I_{y,tf} + I_{y,w} + I_{y,bf}$

  - Torsional constant $J = \frac{1}{3}\left[2ht_w^{\,3} + 2b_f t_f^{\,3} + b_p t_p^{\,3}\right]$

  - Width-to-thickness ratio of top flange $\lambda_{tf} = \frac{\max\{b_{f,out}, b_{f,in}\}}{t_f}$ - Width-to-thickness ratio of web $\lambda_w = \frac{h}{t_w} = \frac{H - t_f - t_p}{t_w}$

  - Width-to-thickness ratio of bottom flange $\lambda_{bf} = \frac{b_p}{t_p}$

- U-shaped steel section with concrete infill

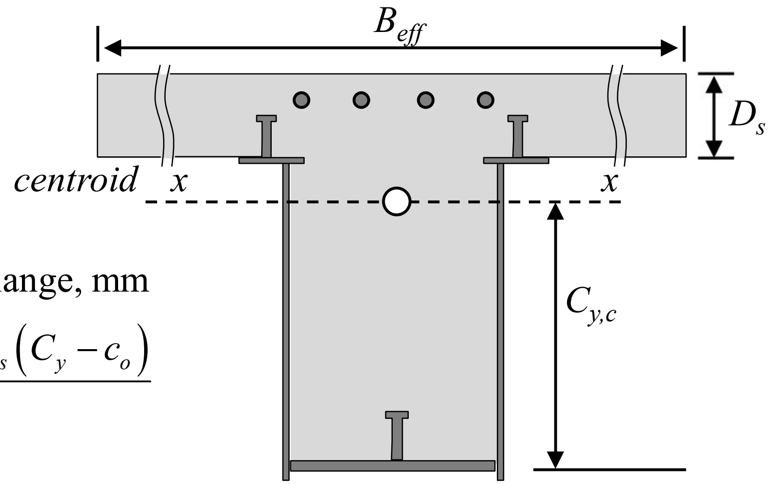
  - Area of slab reinforcements A_{sr}

  - Distance from the centroid of composite section to the outer surface of bottom flange, mm

$$C_{y,c} = \frac{B_{eff}D_s E_c/E_s \times \left(H + D_s/2 - c_o\right) + b_p\left(h - c_o - t_p\right)E_c/E_s \times \left(h/2 - c_o/2\right) + A_s\left(C_y - c_o\right)}{B_{eff}D_s E_c/E_s + b_p\left(h - c_o - t_p\right)E_c/E_s + A_s}$$

  - Moment of inertia for the fully composite uncracked transformed section

$$I_{tr} = B_{eff}D_s E_c/E_s \times \left(H + D_s/2 - C_{y,c}\right)^2 + b_p\left(h - c_o - t_p\right)E_c/E_s \times \left(\left(h - c_o - t_p\right)/2 + t_p - C_{y,c}\right)^2 + A_s \times \left(C_y - C_{y,c}\right)^2$$



2. Design Procedure

Step 1. Check Limitations (AISC 360-16)

- Material limitations (I1.3.)

  - The compressive strength of concrete: 21 MPa $\leq f_{ck} \leq$ 69 MPa

  - The yield strength of structural steel: $F_y \leq$ 525 MPa

  - The yield strength of reinforcing bars: $F_{yr} \leq$ 550 MPa

- (Construction stage) Classification of steel sections for local buckling

  - Flanges of singly symmetric I-shaped built-up sections (B4 > Table B4.1b > Case 12)

  - Webs of singly symmetric I-shaped sections (B4 > Table B4.1b > Case 16)

- (Composite section) Classification of filled composite sections for local buckling (I4 > Table I1.1b)

- Composite beams with formed steel deck (I3 > 2c.)

  - The concrete slab shall be connected to the steel beam with steel headed stud anchors welded either through the deck or directly to the steel cross section.

  - Steel headed stud anchors, after installation, shall extend not less than <u>38 mm above the top of the steel deck</u> and there shall be at least <u>13 mm of specified concrete cover above the top of the steel headed stud anchors</u>.

  - <u>The slab thickness above the steel deck shall be not less than 50 mm</u>.

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

↓

Check limitations

↓

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

↓

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

↓

End of procedure

2. Design Procedure

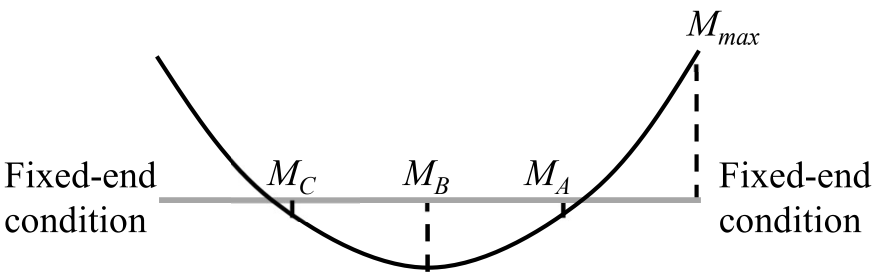
Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(1) General provisions(F1.)

- The reduction factor for the design flexural strength $\Phi_b = 0.9$
- **The lateral-torsional buckling modification factor** for singly symmetric sections in single and reverse curvature.(C-F1-3)

$$C_b = \left(\frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \right) R_M \leq 3.0$$

- M_{max} = Absolute value of moment in the unbraced segment
- M_A = Absolute value of moment at quarter point of the unbraced segment
- M_B = Absolute value of moment at centerline of the unbraced segment
- M_C = Absolute value of moment at three-quarter point of the unbraced segment
- For single curvature bending, $R_m = 1.0$
- For reverse curvature bending, $R_m = 0.5 + 2\left(I_{y,tf} / I_y\right)^2$ (C-F1-4)



Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

2. Design Procedure

Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(2) Positive moment and negative moment

- The flexural capacity of U-shaped steel section is computed according to **F4 (singly symmetric I-shaped members with compact and noncompact webs)** and **F5 (singly symmetric I-shaped members with slender webs)**
- **The nominal flexural strength, M_n,** shall be the lowest value obtained according to the limit states of <u>compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding</u>.
- The top flanges and webs are compression members under positive bending.
- The bottom flanges and webs are compression members under negative bending.
- **Compression flange yielding $M_n = R_{pc} M_{yc}$**

  - Yield moment in the compression flange $M_{yc} = F_y S_{xc}$ (F4-1)

  - The web plastification factor R_{pc} is determined as follows:

    when $I_{yc}/I_y > 0.23$

    when $h_c/t_w \le \lambda_{pw}$, $R_{pc} = M_p/M_{yc}$ (F4-9a)

    when $h_c/t_w > \lambda_{pw}$, $R_{pc} = \left[\dfrac{M_p}{M_{yc}} - \left(\dfrac{M_p}{M_{yc}} - 1\right)\left(\dfrac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \dfrac{M_p}{M_{yc}}$ (F4-9b)

    when $I_{yc}/I_y \le 0.23$

    $R_{pc} = 1.0$ (F4-10) where $M_p = F_y Z_x \le 1.6 F_y S_x$

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

2. Design Procedure

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Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(2) Positive moment and negative moment

- **Lateral-torsional buckling**(F4.2.)

  - L_b : length between points that are either braced against lateral displacement of the compression flange or braced against twice of the cross section.

  - L_p : the limiting laterally unraced length for the limit state of yielding $L_p = 1.1 r_t \sqrt{\dfrac{E}{F_y}}$ (F4-7)

  - L_r : the limiting unbraced length for the limit state of inelastic lateral-torsional buckling

$$L_r = 1.95 r_t \frac{E}{F_y} \sqrt{\frac{J}{S_{xc} h_o} + \sqrt{\left(\frac{J}{S_{sc} h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \quad \text{(F4-8)}$$

  - When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

  - When $L_p < L_b \le L_r$,

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{sc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_{pc} M_{yc} \quad \text{(F4-2)}$$

  - When $L_b > L_r$,

$$M_n = F_{cr} S_{sc} \le R_{pc} M_{yc} \quad \text{(F4-3)}$$

  - where,

  The yield moment in the compression flange is $M_{yc} = F_y S_{sc}$ (F4-4)

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

2. Design Procedure

Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(2) Positive moment and negative moment

- **Lateral-torsional buckling**(F4.2.)

- where,

The critical stress $F_{cr} = \dfrac{C_b \pi^2 E}{(L_b / r_t)^2}\sqrt{1 + 0.078 \dfrac{J}{S_{xc} h_o}\left(\dfrac{L_b}{r_t}\right)^2}$ (F4-5)

For $I_{yc} / I_y \leq 0.23$, J shall be taken as zero,

where, I_{yc} = moment of inertia of the compression flange about the y-axis.

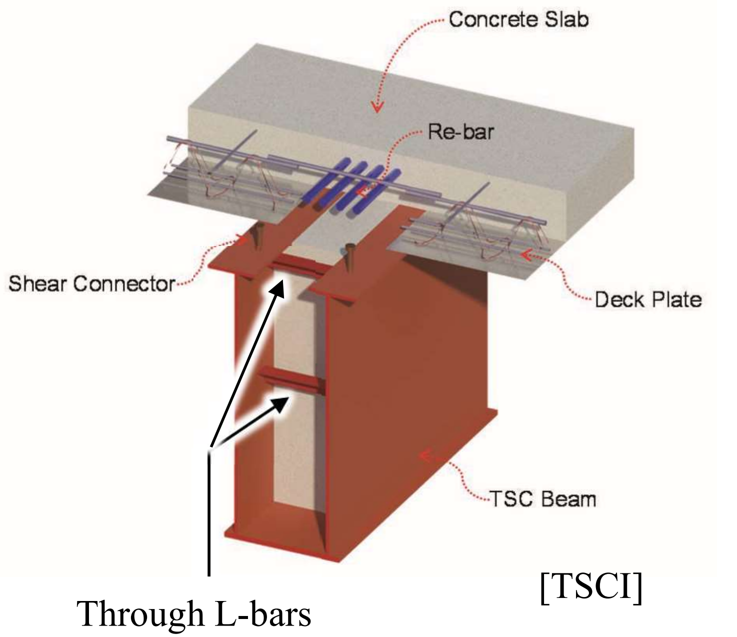
Nominal compression flange stress above which the inelastic buckling limit states

When $S_{xt} / S_{xc} \geq 0.7$, $F_L = 0.7 F_y$ (F4-6a)

When $S_{xt} / S_{xc} < 0.7$, $F_L = F_y S_{xt} / S_{xc} \geq 0.5 F_y$ (F4-6b)

where, S_{xt} = elastic section modulus referred to tension flange.

Note that the lateral-torsional buckling resistance of the TSC beam is determined in accordance with the singly symmetric I-shaped members (i.e. opened steel section) in AISC 360-16, for a conservative method. The torsional constant, J, of the U-shaped steel section, is taken as the sum of the torsional constant of each steel section. The angle reinforcements welded to webs are not considered. <u>In the actual TSC beam, the LTB resistance may be enhanced due to the additional reinforcements such as through L-bars, butterfly-shaped ribs, and deck plates.</u>



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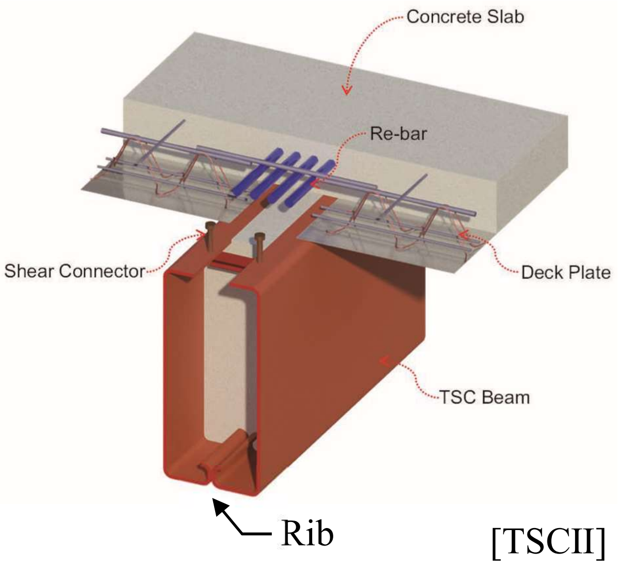

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2. Design Procedure

Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(2) Positive moment and negative moment

- **Compression flange local buckling** (F4.3.)

$$M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$$ for sections with noncompact flanges (F4-13)

$$M_n = \frac{0.9Ek_c S_{xc}}{\lambda^2}$$ for sections with slender flanges (F4-14)

- Nominal compression flange stress above which the inelastic buckling limit state apply

F_L: (When $S_{xt}/S_{xc} \geq 0.7$) $F_L = 0.7F_y$ (F4-6a), (When $S_{xt}/S_{xc} < 0.7$) $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ (F4-6b)

- The width-to-thickness ratio of compression flange λ (Table B4.1b)

- The limiting slenderness for a compact flange λ_{pf} (Table B4.1b)

- The limiting slenderness for a noncompact flange λ_{rf} (Table B4.1b)

- **Tension flange yielding** (F4.4.)

When $S_{xt} < S_{xc}$, $M_n = R_{pt}M_{yt}$ (F4-15)

- Yield moment in the tension flange $M_{yt} = F_y S_{xt}$

- The web plastification factor corresponding to the tension flange yielding limit state R_{pt}

$I_{yc}/I_y > 0.23$ ➔

when $h_c/t_w \leq \lambda_{pw}$, $R_{pt} = M_p/M_{yt}$ (F4-16a)

when $h_c/t_w > \lambda_{pw}$, $$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yt}}$$ (F4-16b)

$I_{yc}/I_y \leq 0.23$ ➔ $R_{pt} = 1.0$ (F4-17)

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

2. Design Procedure

Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(3) Shear strength (G. Design of members for shear)

- For the TSC beam, nominal shear strength, V_n, is determined in accordance with G2.I-shaped members and G2.2.1(b). In general, transverse stiffeners are not considered in the calculation of web plate shear buckling coefficient, k_v.

- General provisions (G1.)

  - The design shear strength, $\Phi_v V_n$
  - $\Phi_v = 0.90$ (LRFD)

- I-shaped members and channels (G2.)

  - Shear strength of webs without tension filed action (G2.1.):

    The nominal shear strength, V_n, is: $V_n = 0.6F_y A_w C_v$

    where, A_w = area of web, the overall depth times the web thickness (mm^2)

  - The web shear strength coefficient, C_v (G2.2.1.(b) For other I-shaped members):

    (i) when $h/t_w \le 1.10\sqrt{k_v E/F_y}$, $C_v = 1.0$ (G2-3)

    where h = for build-up welded sections, the clear distance between flanges

    (ii) when $h/t_w > 1.10\sqrt{k_v E/F_y}$, $C_v = 1.10\sqrt{k_v E/F_y}\,/\,(h/t_w)$ (G2-4)

  - The web plate shear buckling coefficient, k_v, is determined as follows:

    (i) For without transverse stiffeners, $k_v = 5.34$

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

↓

Check limitations

↓

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

↓

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

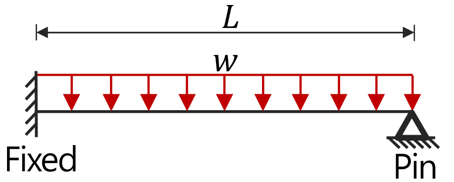
↓

End of procedure

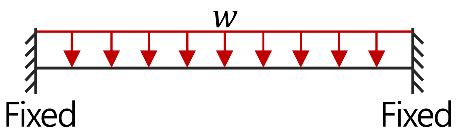
Step 2. Construction Stage (AISC 360-16 F. Design of members for flexure)

(4) Deflection (I3.1b. Strength during construction)

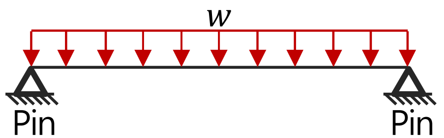
- **The self-weight of concrete for slab and beam, W_d and W_s, is considered as the applied load during concrete curing.**
- **Deflection**:



$$\delta_{const} = \frac{W_d B_{ay} + W_s}{185 E_s I_x} L^4 < \frac{L}{360}$$



$$\delta_{const} = \frac{W_d B_{ay} + W_s}{384 E_s I_x} L^4 < \frac{L}{360}$$



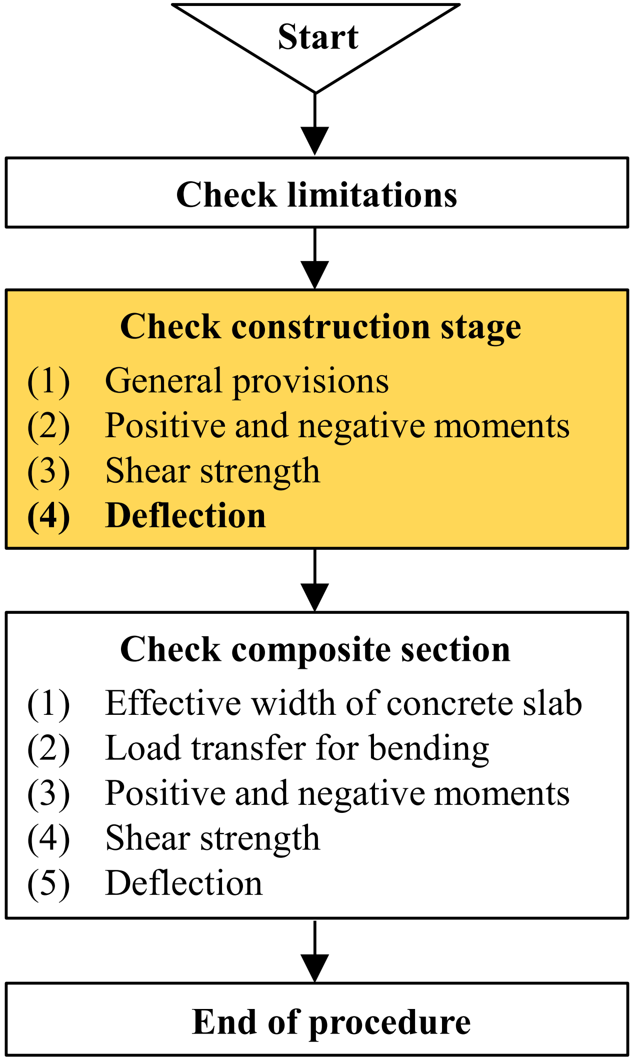
$$\delta_{const} = 5\frac{W_d B_{ay} + W_s}{384 E_s I_x} L^4 < \frac{L}{360}$$

- Live load: TSC beam weight W_s, Slab weight W_d

- The deflection limit for construction load (live load) is determined in accordance with IBC2016 Table 1604.3 Deflection limit as follows:

CONSTRUCTION	L	S or W^f	$D + L^{d,g}$
Roof members:[e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections



2. Design Procedure

SEN ENGINEERING GROUP

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(1) Effective width of concrete slab (I3.1a.)

- The effective width of the concrete slab shall be the sum of the effective widths for each side of the beam centerlines, each of which shall not exceed:

(a) one-eight of the beam span, center-to-center of supports

(b) one-half the distance to the centerline of the adjacent beam

(c) the distance to the edge of the slab

(2) Load transfer for positive flexural strength (I.2d.1.)

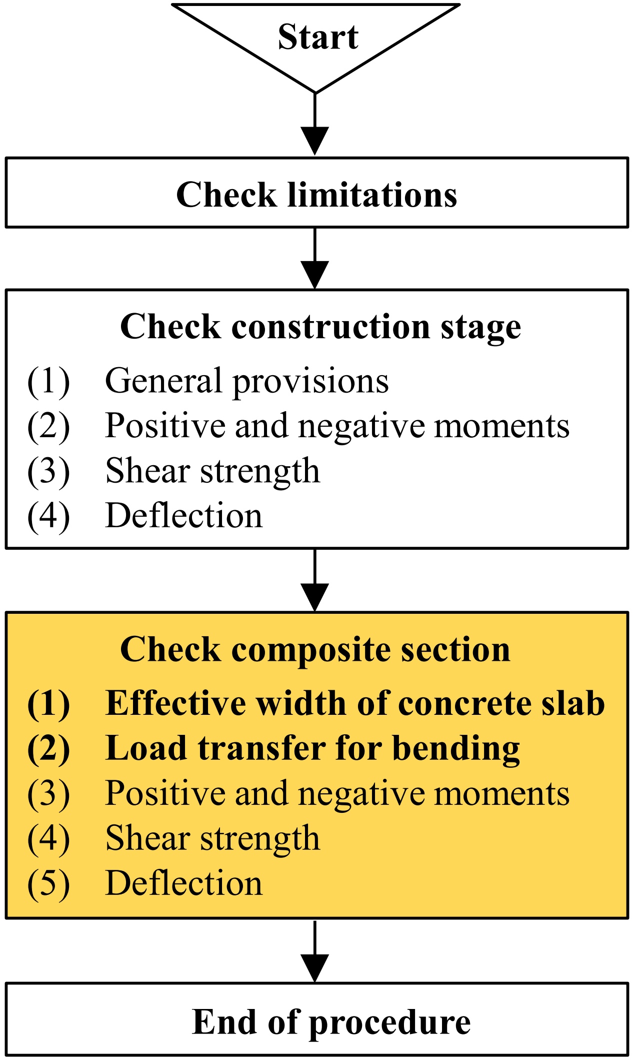
- The nominal shear force between the steel beam and the concrete slab transferred by steel anchors, V', between the point of maximum positive moment and the point of zero moment shall be determined as the lowest value in accordance with the limit states of concrete crushing, tensile yielding of the steel section, or the shear strength of the steel anchors:

(a) Concrete crushing: $V' = 0.85 f_c' A_c$ (I3-1a)

(b) Tensile yielding of the steel section: $V' = F_y A_s$ (I3-1b)

(c) Shear strength of the steel headed stud or steel channel anchors: $V' = \sum Q_n$ (I3-1c)

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections



2. Design Procedure

SEN ENGINEERING GROUP

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(2) Load transfer for negative flexural strength (I.2d.2.)

- In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear between the point of maximum negative moment an the point of zero moment shall be determined as the lower in accordance with the following limit states:

(a) For the limit state of tensile yielding of the slab reinforcement

$V' = F_{ysr} A_{sr}$ (I3-2a)

where, A_{sr} = area of developed longitudinal reinforcing steel within the effective width of the concrete slab

(b) For the limit state of shear strength of steel headed stud, $V' = \sum Q_n$ (I3-2b)

(3) Flexural strength (I4 > **Filled composite members** > I4b.Flexural strength)

- The available flexural strength of filled composite members shall be determined as follows: $\Phi_b = 0.90$ (LRFD)

The nominal flexural strength, Mn, shall be determined as follows:

For compact sections $M_n = M_p$

For noncompact sections $M_n = M_p - (M_p - M_y)(\lambda - \lambda_p)/(\lambda_r - \lambda_p)$

For slender sections, M_n, shall be determined as the first yield moment. The compression flange stress shall be limited to the local buckling stress, F_{cr}.

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

↓

Check limitations

↓

Check construction stage

(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

↓

Check composite section

(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

↓

End of procedure

2. Design Procedure

SEN ENGINEERING GROUP

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-① Positive moment strength (I4. Filled Composite Members)

- The TSC beams are filled with concrete; thus the web and bottom flange sections are classified using limitations of width-to-thickness ratios of walls of rectangular Box sections specified in Table I1.1b.

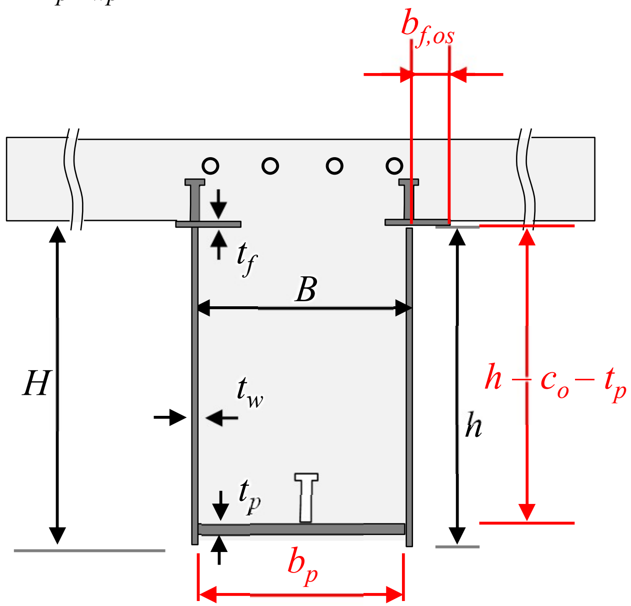
(a) Top flange : width-to-thickness ratio $\lambda_{tf} = b_{f,os}/t_f$

- $\lambda_p \le 0.38\sqrt{E/F_y}$, $\lambda_r \le 1.00\sqrt{E/F_y}$ (Table B4.1b)

(b) Web and bottom flange ($\lambda_w = (h\text{-}c_o\text{-}t_p)/t_w$; $\lambda_{bf} = b_p/t_{wp}$)

- $\lambda \le 3.00\sqrt{E/F_y}$: Compact section
- $3.00\sqrt{E/F_y} < \lambda \le 5.70\sqrt{E/F_y}$

  : Noncompact section
- $\lambda > 5.70\sqrt{E/F_y}$: Slender section



Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

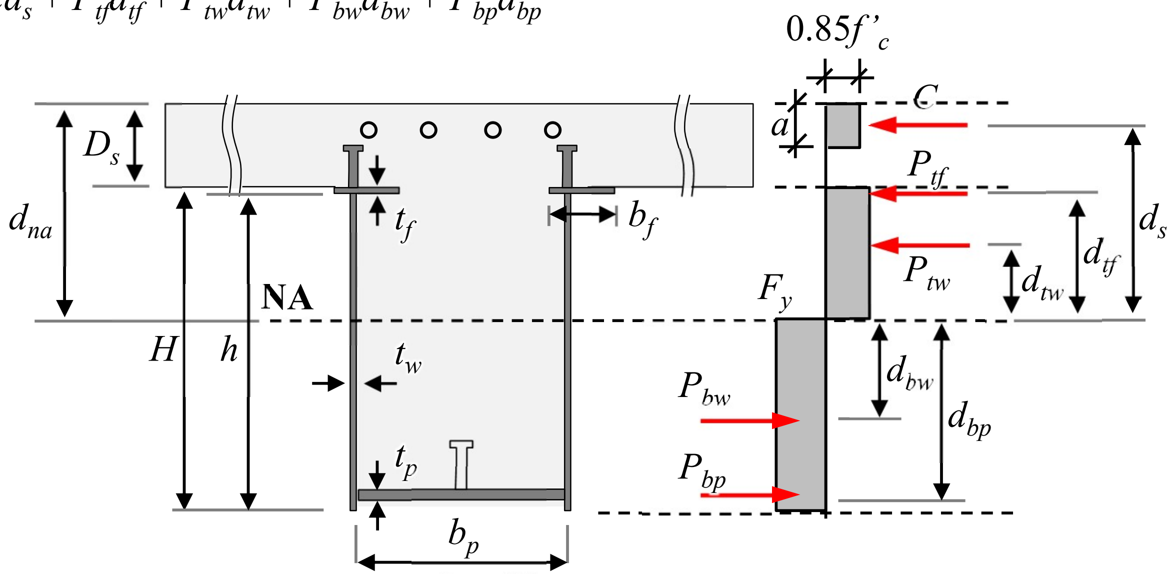
(3)-① Positive moment strength (I4b. Flexural strength)

- The compression force, C, in the concrete slab is smallest of: $C = A_s F_y$ (C-I3-6) $C = 0.85 f_c' A_c$ (C-I3-7) $C = \sum Q_n$ (C-I3-8)
- Longitudinal slab reinforcement makes a negligible contribution to the compression force.
- The depth of the compression block is: $a = C / \left(0.85 f_c' B_{eff}\right)$
- The plastic stress distribution may have the plastic neutral axis, PNA, in the web, in the top flange of the steel section, or in the slab, depending on the governing C. In general, PNA of composite beam is located in the slab or top flange when the full composite action between slab and steel section can be expected. However, in the case of partial composite member (compression force governed by headed studs), PNA is located in the web.
- (Compact section) When PNA is located in the web, $M_p = Cd_s + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bp}d_{bp}$

  - yield strength of the top flange $P_{tf} = 2 \times F_y b_f t_f$
  - yield strength of the web in compression

    $P_{tw} = 2 \times F_y (d_{na} - D_s - t_f) t_w$
  - yield strength of the web in tension

    $P_{bw} = 2 \times F_y (H + D_s - d_{na}) t_w$
  - yield strength of the bottom flange $P_{bp} = F_y b_p t_p$



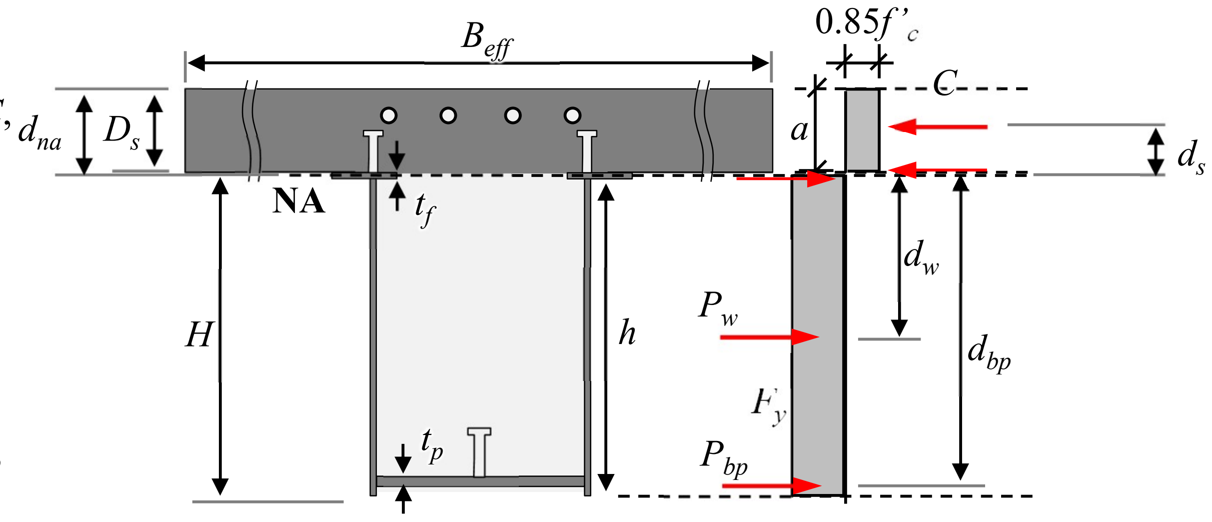
<Stress distribution when PNA is located in the web of TSC beam>

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-① Positive moment strength (I4b. Flexural strength)

- distance from the centroid of the compression force, C, in the concrete to NA; $d_s = d_{na} - 0.5a$
- distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - t_f/2$
- distance from P_{tw} to NA; $d_{tw} = (d_{na} - D_s - t_f)/2$
- distance from P_{bw} to NA; $d_{bw} = (H + D_s - d_{na})/2$
- distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - t_p/2 - c_o$



<Stress distribution when PNA is located in the top flange of TSC beam>

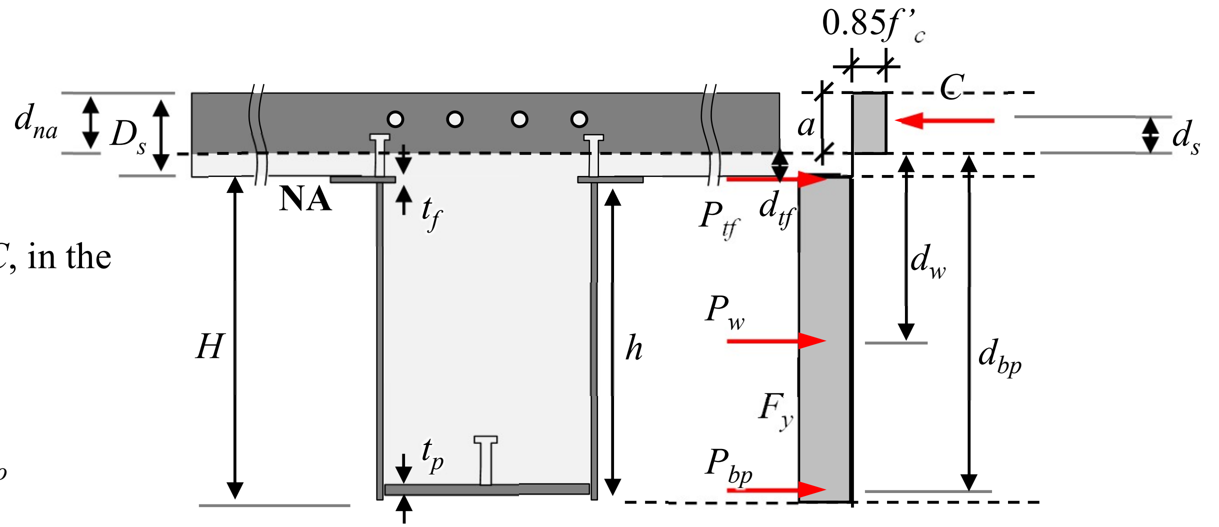
- When PNA is located in the top flange,

$$M_p = Cd_s + P_w d_w + P_{bp} d_{bp}$$

- yield strength of the entire section of web

$$P_w = 2 \times F_y h t_w$$

- yield strength of the bottom flange $P_{bp} = F_y b_p t_p$
- distance from the centroid of the compression force, C, in the concrete to NA; $d_s = d_{na} - 0.5a$
- distance from P_w to NA; $d_w = D_s + t_f + h/2 - d_{na}$
- distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - t_p/2 - c_o$



<Stress distribution when PNA is located in the slab of TSC beam>

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-① Positive moment strength (I4b. Flexural strength)

- When PNA is located in the concrete slab,

$M_p = Cd_s + P_{tf}d_{tf} + P_w d_w + P_{bp}d_{bp}$

- yield strength of the top flange $P_{tf} = 2\times F_y b_f t_f$
- yield strength of the entire section of web $P_w = 2\times F_y h t_w$
- yield strength of the bottom flange $P_{bp} = F_y b_p t_p$
- distance from P_{tf} to NA; $d_{tf} = D_s + t_f/2 - d_{na}$
- distance from P_w to NA; $d_w = D_s + t_f + h/2 - d_{na}$
- distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - t_p/2 - c_o$
- distance from the centroid of the compression force, C, in the concrete to NA; $d_s = 0.5a$

(3)-①* Positive moment strength (I4b. Flexural strength)

- Yield strength M_y for noncompact section;

$M_y = Cd_s + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bp}d_{bp}$

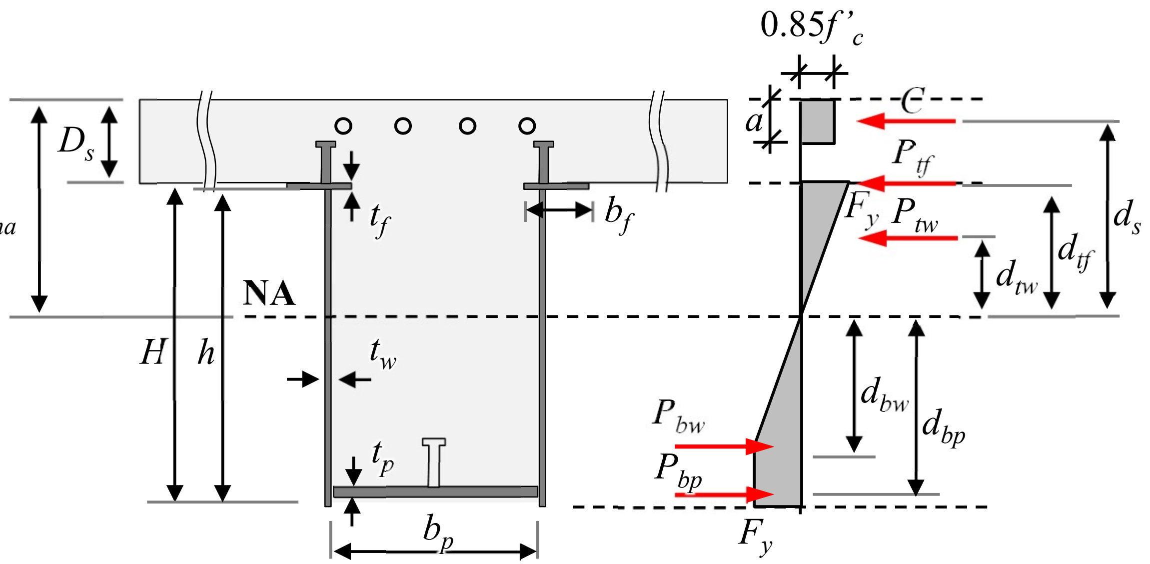
- yield strength of the top flange $P_{tf} = 2\times F_y b_f t_f$
- yield strength of the top web

$P_{tw} = 2\times 0.5\times F_y(d_{na} - D_s - t_f)t_w$

- yield strength of the bottom web

$P_{bw} = 2\times 0.5\times F_y(d_{na} - D_s - t_f)t_w + 2\times F_y[(H + D_s - d_{na}) - (d_{na} - D_s - t_f)]t_w$

- yield strength of the bottom flange $P_{bp} = b_p t_p F_y$



<Stress distributions for calculating M_y - Noncompact section>

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-① * Positive moment strength (I4b. Flexural strength)

- distance from the centroid of the compression force, C, in the concrete to PNA; $d_s = d_{na} - 0.5a$
- distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - t_f/2$
- distance from P_{tw} to NA; $d_{tw} = 2/3 \times (d_{na} - D_s - t_f)$
- distance from the center of web tension to PNA; $d_{bw} = h - (d_{na} - D_s - t_f) - P_{bw}/(4F_y t_w)$
- distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - t_p/2 - c_o$

- First yield strength M_{cr} for slender section;

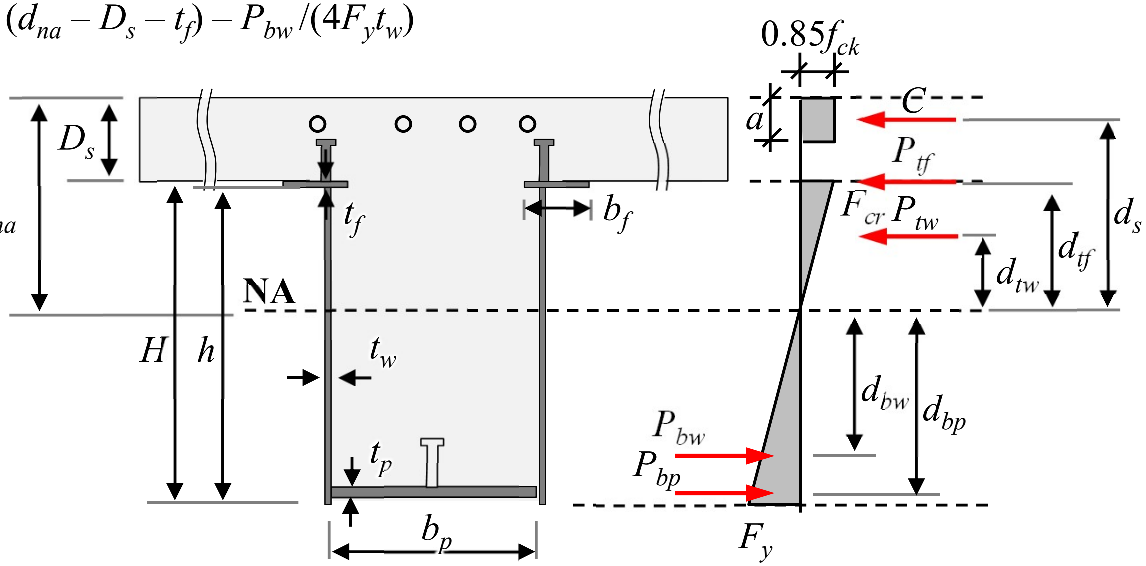
$$M_{cr} = Cd_s + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bp}d_{bp}$$

- yield strength of the top flange $P_{tf} = 2 \times F_{cr}\, b_f t_f$
- yield strength of the top web in compression

$$P_{tw} = F_{cr}(d_{na} - D_s - t_f)t_w$$

- yield strength of the top web in tension

$$P_{bw} = F_y(H + D_s - d_{na})t_w$$



<Stress distributions for first yield moment, M_{cr} – Slender section>

- yield strength of the bottom flange $P_{bp} = F_y b_p t_p$
- distance from the centroid of the compression force, C, in the concrete to NA; $d_s = d_{na} - 0.5a$
- distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - t_f/2$
- distance from P_{tw} to NA; $d_{tw} = 2(d_{na} - D_s - t_f)/3$
- distance from P_{bw} to NA; $d_{bw} = 2(H + D_s - d_{na})/3$
- distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - t_p/2 - c_o$

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-② Negative moment strength (I4b. Flexural strength)

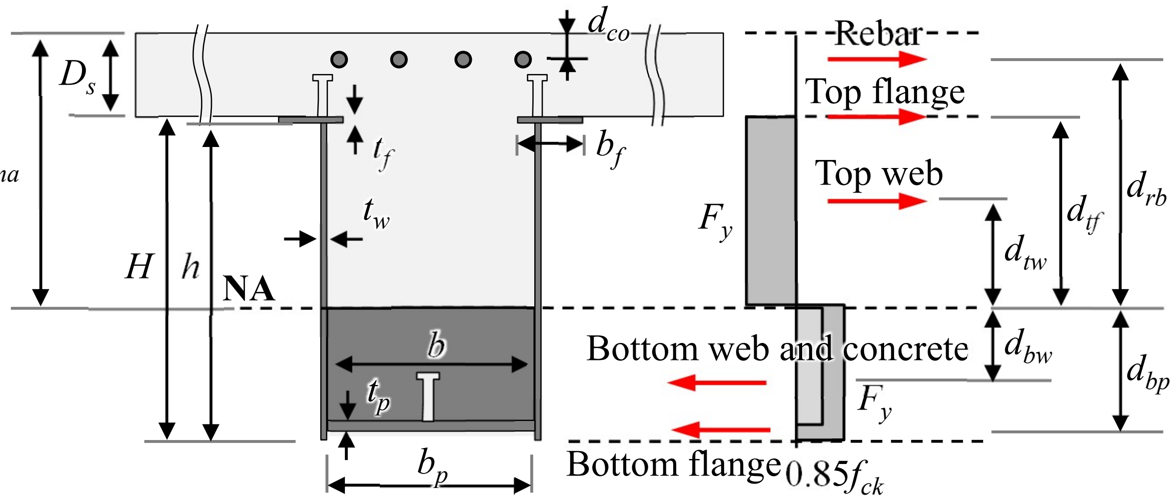
- This section applies to three types of composite members subject to flexure: composite beams with steel anchors consisting of steel headed stud anchors or steel channel anchors, concrete encased members, and **concrete filled members**. (I3. Flexure)
- When an adequately braced **compact steel section** and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the **plastic distribution**. (comment 2b. Negative flexural strength)
- **Compact section classification** of TSC webs and bottom flange under compression:

  (Web) $h/t_w \leq 3.00\sqrt{E/F_y}$ (bottom flange) $b/t_p \leq 2.26\sqrt{E/F_y}$
- The tensile force, T, in the reinforcing bars is the smaller of: $T = F_{yr}A_r$ (C-I3-11), $T = \sum Q_n$ (C-I3-12)

  where, A_r = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, F_{yr} = specified minimum yield stress of the slab reinforcement, $\sum Q_n$ = sum of the nominal strengths of steel headed anchors between the point of maximum negative moment and the point of zero moment to either side.
- For the compact section,

  $M_p = Td_{rb} + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bc}d_{bw} + P_{bp}d_{bp}$

  - tensile force $T = \min\{F_{yr}A_r, \sum Q_n\}$
  - yield strength of the top flange $P_{tf} = 2\times F_y b_f t_f$
  - yield strength of the top web $P_{tw} = 2\times F_y(d_{na} - D_s - t_f)$



2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-② Negative moment strength (I4b. Flexural strength)

- Yield strength of bottom web $P_{bw} = 2 \times F_y (H + D_s - d_{na}) t_w$
- Compressive strength bottom concrete $P_{bc} = 0.85 f'_c (H + D_s - d_{na}) b$
- Yield strength of bottom flange $P_{bp} = F_y b_p t_p$
- Distance from T to NA; $d_{rb} = d_{na} - d_{co}$

- Distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - 0.5 t_f$
- Distance from P_{tw} to NA; $d_{tw} = (d_{na} - D_s - t_f)/2$
- Distance from P_{bw} to NA; $d_{bw} = (H + D_s - d_{na})/2$
- Distance from P_{bc} to NA; $d_{bw} = (H + D_s - d_{na} - t_p - c_o)/2$
- Distance from P_{bp} to NA; $d_{bp} = H + D_s - d_{na} - 0.5 t_p - c_o$

(3)-②* Negative moment strengths for noncompact and slender sections

- Width-to-thickness ratio of webs (table I1.1b)

  $3.00\sqrt{E/F_y} < h/t_w \le 5.70\sqrt{E/F_y}$ ➔ Noncompact s.

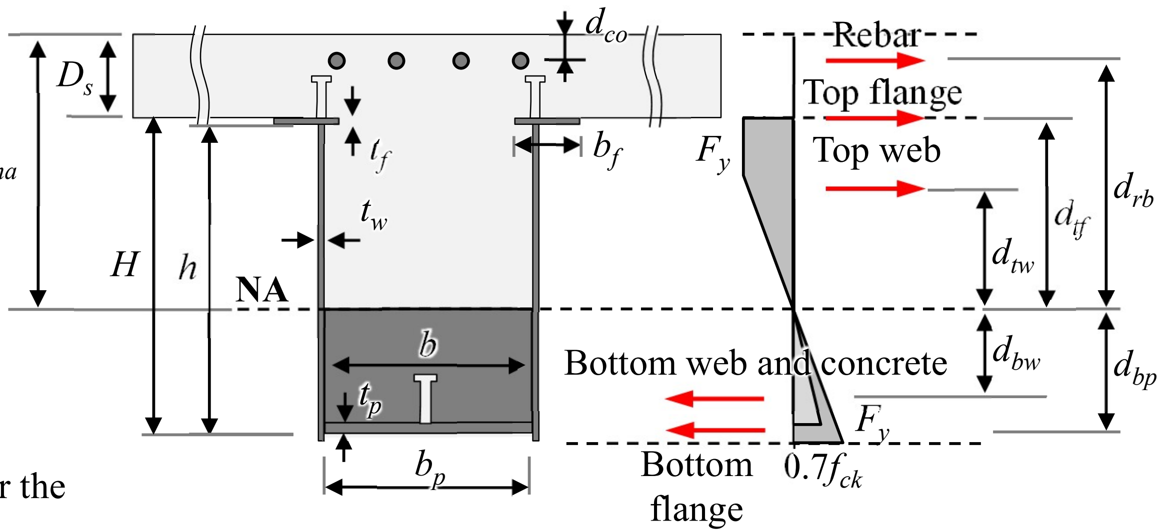
  $5.70\sqrt{E/F_y} < h/t_w$ ➔ Slender section

- Width-to-thickness ratio of bottom flange (table I1.1b)

  $2.26\sqrt{E/F_y} < b/t_p \le 3.00\sqrt{E/F_y}$ ➔ Noncompact s.

  $3.00\sqrt{E/F_y} < b/t_p$ ➔ Slender section

- If the width-to-thickness ratio of webs $h/t_w > 3.00\sqrt{E/F_y}$ for the bottom flange is classified as noncompact section, M_n is determined from the elastic stresses, considering yield moment M_y.
- If the bottom flange is classifies as slender section, M_n is determined from the elastic stresses, considering first buckling strength moment M_{cr}.



<Stress distribution for calculating M_y>

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

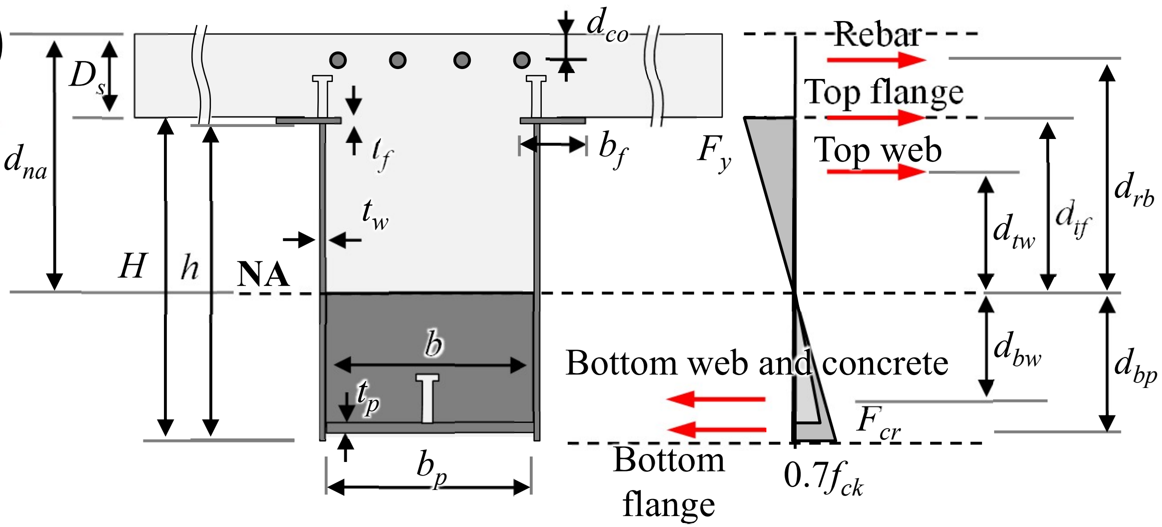
(3)-②* Negative moment strengths for noncompact and slender sections

- (Noncompact section)Yield strength $M_y = Td_{rb} + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bc}d_{bw} + P_{bp}d_{bp}$
  - tensile force $T = \min\{F_{yr}A_r, \sum Q_n\}$
  - Yield strength of the top web $P_{tw} = 2F_y\left(d_{na} - D_s - t_f - d_{bp}\right)t_w + F_y d_{bf} t_w$
  - yield strength of the top flange $P_{tf} = 2 \times F_y b_f t_f$
  - Yield strength of the bottom web $P_{bw} = F_y\left(d_{bp} - t_p / 2\right)t_w$
  - Compressive strength of bottom concrete $P_{bc} = 0.7 f_{ck}\left(d_{bp} - t_p / 2\right)b / 2$
  - Yield strength of the bottom flange $P_{bp} = F_y b_p t_p$
  - Distance from T to NA; $d_{rb} = d_{na} - d_{co}$
  - Distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - t_f / 2$
  - Distance from P_{tw} to NA; $d_{tw} = d_{na} - D_s - t_f - P_{tw} / \left(4F_y t_w\right)$
  - Distance from P_{bw} to NA; $d_{bw} = 2/3\left(H + D_s - d_{na}\right)$
  - Distance from P_{bc} to NA; $d_{bc} = 2/3\left(H + D_s - d_{na} - c_o - t_p\right)$
  - Distance from P_{bf} and to NA; $d_{bp} = H + D_s - t_p / 2 - d_{na} - c_o$

- (Slender section) First yield strength

$$M_{cr} = Td_{rb} + P_{tf}d_{tf} + P_{tw}d_{tw} + P_{bw}d_{bw} + P_{bc}d_{bw} + P_{bp}d_{bp}$$

  - Critical buckling strength of bottom flange (I2-10; buckling strength for rectangular filled section) $F_{cr,bf} = 9E_s / \left(b / t_p\right)^2$
  - Tensile force $T = \min\{F_{yr}A_r, \sum Q_n\}$
  - Yield strength of the top flange $P_{tf} = 2 \times F_y b_f t_f$



<Stress distribution for slender section>

2. Design Procedure

Step 3. Check Composite Section (AISC 360-16 I. Design of composite members)

(3)-②* Negative moment strengths for noncompact and slender sections

- (Slender section) First yield strength

  - Yield strength of the top web $P_{tw} = F_y\left(d_{na} - D_s - t_f\right)t_w$

  - Yield strength of the bottom web $P_{bw} = F_{cr}\left(H + D_s - d_{na}\right)t_w$

  - Compressive strength of the concrete $P_{bc} = 0.7 f_{ck} b\left(H + D_s - d_{na} - c_o - t_p\right)/2$

  - Yield strength of the bottom flange $P_{bf} = F_{cr} b_p t_p$

  - Distance from T to NA; $d_{rb} = d_{na} - d_{co}$

  - Distance from P_{tf} to NA; $d_{tf} = d_{na} - D_s - t_f/2$

  - Distance from P_{tw} to NA; $d_{tw} = 2/3\left(d_{na} - D_s - t_f\right)$

  - Distance from P_{bw} to NA; $d_{bw} = 2/3\left(H + D_s - d_{na}\right)$

  - Distance from P_{bc} to NA; $d_{bw} = 2/3\left(H + D_s - d_{na} - c_o - t_p\right)$

  - Distance from P_{bf} to NA; $d_{bp} = H + D_s - t_p/2 - d_{na}$

(4) The nominal shear strength (I.4.2. Composite beams with formed steel deck)

- The design shear strength $\Phi_v V_n$ shall be determined based on the available shear strength of steel section alone as specified in Chapter G.

  - The nominal shear strength, V_n, is: $V_n = 0.6F_y A_w C_v$

Given: Beam geometry, required shear and moment strengths, deflection limit
Find: nominal shear and moment strengths, deflection for steel and composite sections

Start

Check limitations

Check construction stage
(1) General provisions
(2) Positive and negative moments
(3) Shear strength
(4) Deflection

Check composite section
(1) Effective width of concrete slab
(2) Load transfer for bending
(3) Positive and negative moments
(4) Shear strength
(5) Deflection

End of procedure

3. Example (Calculation sheet TG1)

3. Calculation sheet (TG1)

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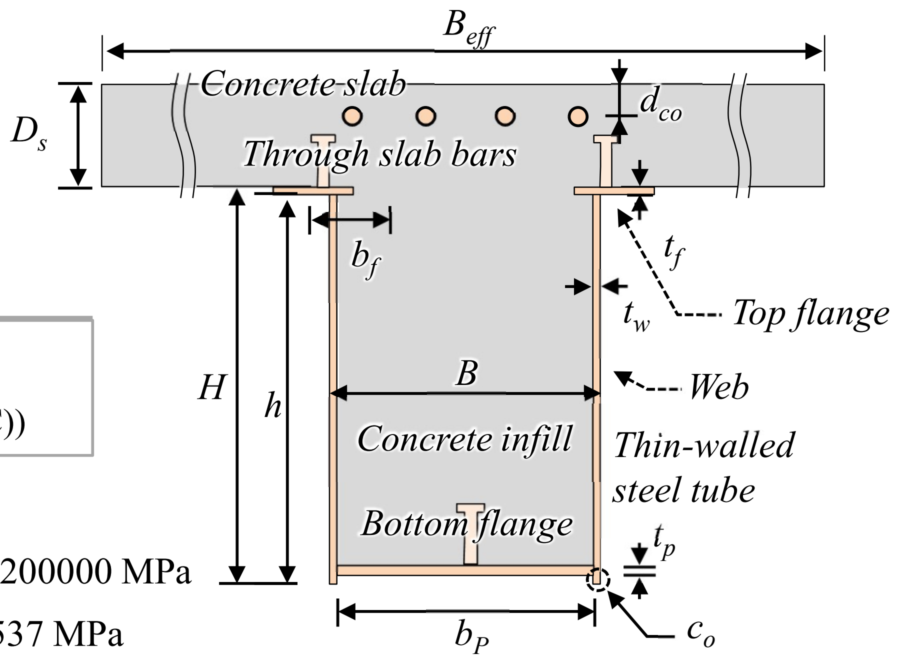
1.Design Conditions > 1.1 Material Properties



- Steel plate for SM355 grade
  - Specified yield strength of steel plate F_y = 355 MPa
  - Modulus of elasticity E_s = 210000 MPa

> (KS D 3515) SM490A-Rolled steels for welded structures similar to A572 GR. 50
>
> - SM: Steel marine (plate/rolled H); 490: Tensile strength; A: CVN grade (≥ 27J(20˚C))

- Headed stud anchor (shear connector): Tensile strength F_u = 400 MPa
- Reinforcement bar: Yield strength F_{yr} = 600 MPa; Modulus of elasticity E_r = 200000 MPa
- Concrete : Compressive strength f_{ck} = 30 MPa, Modulus of elasticity E_c = 27537 MPa

> (AISC 360-16) $E_c = 0.043 w_c^{1.5}\sqrt{f'_c}$ = 13682 ~ 29440 MPa
>
> where f'_c = specified compressive strength of concrete (30 MPa) and w_c = weight of concrete per unit volume (1500 ≤ w_c ≤ 2500 kg/m^3)
>
> (AISC 360-16; I1.3 Material limitations) For the determination of the available strength, concrete shall have a compressive strength, f'_c, of not less than 21 MPa nor more than 69 MPa for normal weight concrete.

1. Design Conditions > 1.2 TSC Beam Conditions

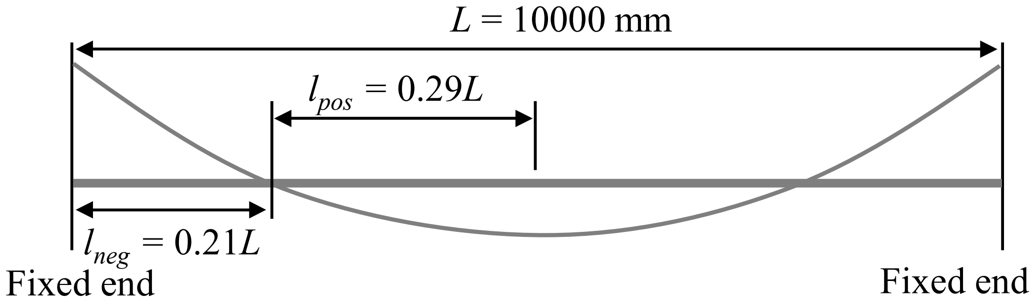
- Dimensions: U-600×400×9×12; H = 600 mm; B = 400 mm; b_f = 120 mm; b_p = 382 mm; t_f = 9 mm; t_w = 9 mm, t_p = 12 mm, c_o = 15 mm
- Top slab bars: 4-D25 (A_{sr} = 1962 mm^2); concrete cover d_{co} = 40 mm
- Slab thickness D_s = 200 mm
- Beam length L = 10000 mm; Beam spacing B_{ay} = 3000 mm

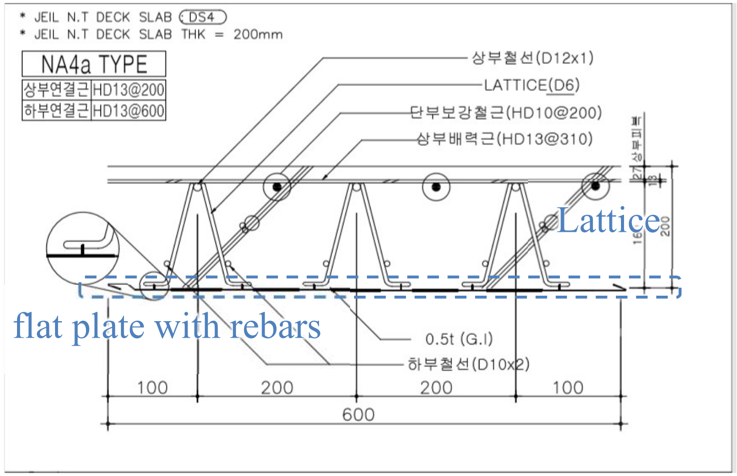
3. Calculation sheet (TG1)

1. Design Conditions > 1.1 Material Properties > (3) Shear connector

- **Shear connector**: **2Row-Φ19@200**; height of the stud h_s = 80 mm; $D_{snet} - h_s$ = 120 mm

  → **spacing 200 mm > 6×d_h**





<Deck plate with flat plate and lattice>

AISC 360-16 > I8. Steel anchors > 2. Steel anchors in composite beams > 2d. Detailing requirements

- Steel anchors required on each side of the point of maximum bending moment, positive or negative, <u>shall be distributed uniformly between that point and the adjacent points of zero moment</u>.
- The minimum distance from the center of a steel anchor to a free edge in the direction of the shear force shall be 200 mm (normal weight conc.).
- Minimum center-to-center spacing of steel headed stud anchors shall be <u>four diameters in any direction</u>. For composite beams that do not contain anchors located within formed steel deck oriented perpendicular to the beam span, an additional minimum spacing limit of six diameters along the longitudinal axis of the beam shall apply.

(AISC 360-16 > I3. Flexure > 2. Composite beams with steel headed stud > 2c. Composite beams with formed steel deck)

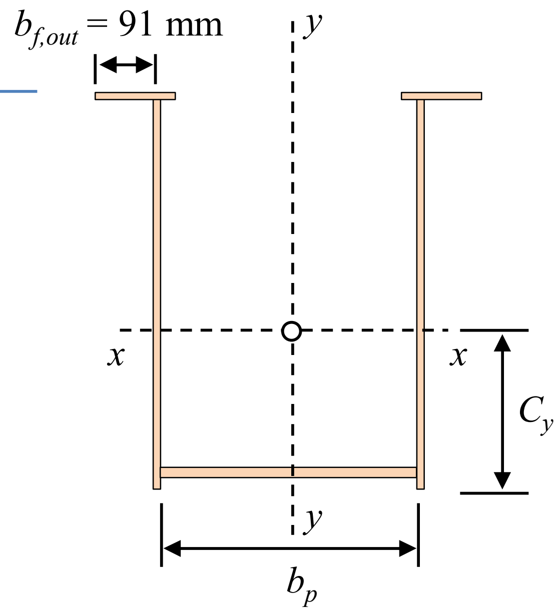
- Limitations for formed steel deck ➔ <u>Deck plate with flap plate and lattice</u> is used for the TSC beam.

3. Calculation sheet (TG1)

1. Design Conditions > 1.4 Section Forces

- Maximum required moment strength under positive moment region, $M_{u,pos}$ = 712 kN·m
- Maximum required moment strength under negative moment region, $M_{u,neg}$ = 1425 kN·m
- Maximum Required shear strength, V_u = 610 kN

Steel section without concrete and tie bars



2. Check Construction Stage > 2.1 TSC Section Properties

- Area of steel section $A_s = 2b_f t_f + 2ht_w + b_p t_p$ = 17382 mm^2
- Distance from the centroid of steel section to the outer surface of bottom flange C_y = 260 mm
- Moment of inertia I_x = 82808 × 10^4 mm^4
- (bottom) Elastic section modulus $S_{x,bf} = I_x / (C_y - c_o)$ = 338 × 10^4 mm^3
- (top) Elastic section modulus $S_{x,tf} = I_x / (H - C_y)$ = 243 × 10^4 mm^3
- Plastic section modulus Z_x = 3396 × 10^3 mm^3

2. Check Construction Stage > 2.2 Width-to-Thickness Ratio

(1) Positive moment (Compression region : top flange and web)

- Top flange (Case 11): $\lambda_{p,tf} = 0.38\sqrt{E_s / F_y} = 9.2 < b_{f,out}/t_f = 10.1 < \lambda_{r,tf} = 0.95\sqrt{k_c E_s / F_L} = 19.5$ ➔ Noncompact section
- Web (Case 16): $\lambda_{pw} = 49.1 < 2(h - {}_bC_y)/t_w = 73.5 < \lambda_{rw} = 5.70\sqrt{E_s / F_y} = 138.6$ ➔ Noncompact section

AISC 360-16 > B4. Member properties > Table B4.1b width-to-thickness ratios: compression elements members subject to flexure

- (Case 11) Flanges of doubly and **singly symmetric I-shaped built-up sections**
- (Case 16) Webs of **singly symmetric I-shaped sections**

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2. Check Construction Stage > 2.2 Width-to-Thickness Ratio

(2) Negative moment (Compression region : bottom flange and web)

- Bottom flange(Case 17): $\lambda_{p,bf} = 1.12\sqrt{E_s / F_y} = 27.2 < b_p / t_p = 31.8 < \lambda_{r,bf} = 1.40\sqrt{E_s / F_y} = 34.0$ ➔ Noncompact section
- Web: $\lambda_{pw} = 49.1 < 2(C_y - t_p - c_o) / t_w = 51.7 < \lambda_{rw} = 5.70\sqrt{E_s / F_y} = 138.6$ ➔ Noncompact section

AISC 360-16 > B4. Member properties > Table B4.1b width-to-thickness ratios: compression elements members subject to flexure

- (Case 17) **Flanges of rectangular HSS**
- (Case 16) Webs of **singly symmetric** I-shaped sections

2. Check Construction Stage > 2.3 Flexural Strength

(1) Construction load (Boundary condition : Fix-Fix)

- Slab weight (Slab thickness = 200 mm), $W_d = 4704$ N/m^2
- Construction load $W_c = 2500$ N/m^2; TSC beam weight $W_s = 6666$ N/m

(2) Positive moment

- Required moment strength $M_{u,pos} = [(1.2W_d + 1.6W_c) \times B_{ay} + 1.2W_s] \times L^2/24 = 153.89$ kN·m
- Nominal flexural strength strength $\Phi M_{nc,pos} = 0.9 \times \min[M_{nc,pos1}, M_{nc,pos2}, M_{nc,pos3}, M_{nc,pos4}]$
  - $M_{nc,pos1}$ = (limit state) compression flange yielding
  - $M_{nc,pos2}$ = (limit state) compression flange local buckling
  - $M_{nc,pos3}$ = (limit state) tension flange yielding
  - $M_{nc,pos4}$ = (limit state) lateral-torsional buckling

3. Calculation sheet (TG1)

2. Check Construction Stage > 2.3 Flexural Strength

AISC 360-16 > F. Design of members for flexure > **F4. Other I-shaped members with compact or noncompact webs bent about their major axis**

- The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

(2) Positive moment (Noncompact flange and web ➔ limit state: $M_{nc,pos2}$; $M_{nc,po3}$; $M_{nc,pos4}$)

- **(top flange) Compression flange local buckling**, $M_{nc,pos2} = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{x,tf}\right)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$ = **792.55** kN·m

  where,

  - Moment of inertia of the compression flange about the y-axis $I_{yc,tf} = 11340 \times 10^4$ mm^4
  - Moment of inertia about the principal axis $I_y = 57580 \times 10^4$ mm^4 ➔ $I_{yc,tf} / I_y = 0.19 \le 0.23$ ➔ $R_{pc} = 1.00$

  - Yield moment in the compression flange $M_{yc} = F_y S_{x,tf} = 355 \times 2430 \times 10^3 = 862.65$ kN·m

  - $\dfrac{S_{x,bf}}{S_{x,tf}} = \dfrac{3380 \times 10^3}{2430 \times 10^3} = 1.39 > 0.7$ ➔ $F_L = 0.7F_y = 248.50$ MPa

- **(bottom flange) Tension flange yielding**, $M_{nc,pos3}$: $S_{x,bf} > S_{x,tf}$ ➔ the limit state of tension flange yielding does not apply

- **Lateral-torsional buckling**, $M_{nc,pos4} = F_{cr} S_{x,tf}$ = **205.3** kN·m

  where,

  - The effective radius of gyration $r_t = \dfrac{b_{fc}}{\sqrt{12\left(1 + a_w / 6\right)}} = 57.37$ mm

    where, width of compression flange $b_{fc} = 2b_{tf} = 240$ mm and $a_w = \dfrac{2\left(H - t_f - C_y\right)t_w}{b_{fc}t_f} = 2.76$

3. Calculation sheet (TG1)

2. Check Construction Stage > 2.3 Flexural Strength

- The limiting laterally unbraced length for the limit state of yielding $L_p = 1.1 r_t \sqrt{\dfrac{E}{F_y}}$ = **1534** mm

- The limiting unbraced length for the limit state of inelastic lateral-torsional buckling

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{x,tf} h_o} + \sqrt{\left(\frac{J}{S_{x,tf} h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} \quad = \mathbf{5599} \text{ mm}$$

where,

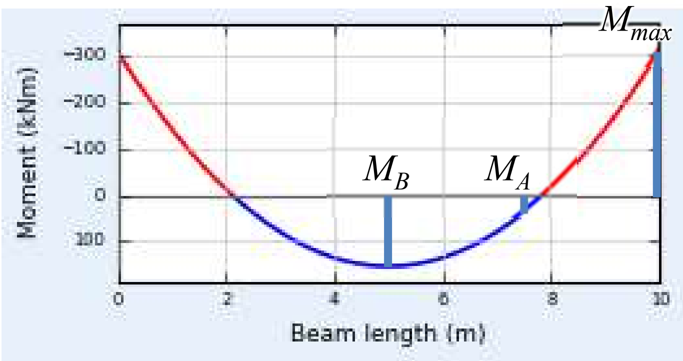
Torsional constant for thin walled open tube $J = \frac{1}{3}\left[2\left(H - t_f\right)t_w^{\ 3} + 2b_f t_f^{\ 3} + b_p t_p^{\ 3}\right]$ = 565578 mm^4

$h_o = H - t_f/2 - t_p/2 - c_o = 574.5$ mm

- L_b = 10000 mm ➔ $L_b > L_r$

- The lateral-torsional buckling modification factor, C_b, for nonuniform moment diagrams when both ends of the segment are braced.

$$C_b = \left(\frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_A + 4M_B + 3M_C}\right) R_M \quad = 1.24 \le 3.0 \quad \text{(C-F1-3)}$$



where,

Absolute value of maximum moment in the unbraced segment M_{max} = 307.78 kN·m

Absolute value of moment at quarter point of the unbraced segment M_A = 38.04 kN·m

Absolute value of moment at centerline of the unbraced segment M_B = 153.89 kN·m

Absolute value of moment at three-quarter point of the unbraced segment M_c = 38.04 kN·m

R_m = 1.0 for single curvature bending and $0.5+2(I_{yc}/I_y)^2$ for reverse curvature bending (= 0.52 refer to Comm. F1.)

3. Calculation sheet (TG1)

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2. Check Construction Stage > 2.3 Flexural Strength

AISC 360-16 > F. Design of members for flexure > **F4. Other I-shaped members with compact or noncompact webs bent about their major axis**

(comment) Kirby and Nethercot present an equation that is a direct fit to various nonlinear moment diagrams within the unbraced segment. This equation gives a more accurate solution for unbraced lengths in which the moment diagram deviates substantially from a straight line, such as the case of a fixed-end beam with no lateral bracing within the span, subjected to a uniformly distributed transverse load. ~ ***The lateral-torsional buckling modification factor given by Equation C-F1-2a is applicable for doubly symmetric sections and singly symmetric sections in reverse curvature.***

- The critical stress $F_{cr} = \dfrac{C_b \pi^2 E}{(L_b / r_t)^2}\sqrt{1 + 0.078\dfrac{J}{S_{x,tf} h_o}\left(\dfrac{L_b}{r_t}\right)^2}$ = 84.5 MPa

➔ $I_{yc,tf} / I_y \leq 0.23$, J is taken as zero

- **Nominal flexural strength $\Phi_b M_{nc,pos4}$ = 0.9×205.3 kN·m = 184.7 kN·m > $M_{u,pos}$ = 153.89 kN·m … O.K.**

(3) Negative moment (Noncompact flange and web ➔ limit state: $M_{nc,neg2}$; $M_{nc,neg3}$; $M_{nc,neg4}$)

- Required moment strength $M_{u,neg} = [(1.2W_d + 1.6W_c)\times B_{ay} + 1.2W_s]\times L^2/12$ = 307.78 kN·m
- **(bottom flange) Compression flange local buckling,** $M_{nc,neg2} = R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{x,bf}\right)\left(\dfrac{\lambda - \lambda_{pp}}{\lambda_{rp} - \lambda_{pp}}\right)$ = **1025.36** kN·m

  where,

  - Moment of inertia of the compression flange about the y-axis $I_{yc,bf}$ = 5574×10^4 mm^4
  - Moment of inertia about the principal axis I_y = 57580×10^4 mm^4 ➔ $I_{yc,bf} / I_y = 0.09 \leq 0.23$ ➔ R_{pc} = 1.00

3. Calculation sheet (TG1)

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2. Check Construction Stage > 2.3 Flexural Strength

- Yield moment in the compression flange $M_{yc} = F_y S_{x,bf} = 355 \times 3380 \times 10^3 = 1199.9$ kN·m
- $\dfrac{S_{x,tf}}{S_{x,bf}} = \dfrac{2430 \times 10^3}{3380 \times 10^3} = 0.72 > 0.7$ ➔ $F_L = 0.7F_y = 248.50$ MPa

Nominal strength

• **(top flange; $S_{x,tf} < S_{x,bf}$) Tension flange yielding**, $M_{nc,neg3} = R_{pt}M_{yt} = F_y S_{x,tf}$ **= 862.6 kN·m** ➔ $\boldsymbol{\Phi_b M_{nc,neg3}}$ **= 776 kN >** $\boldsymbol{M_{u,neg}}$ **… O.K.**

- Moment of inertia of the compression flange about the y-axis $I_{yc,bf} = 5574 \times 10^4$ mm^4
- Moment of inertia about the y-axis $I_y = 57580 \times 10^4$ mm^4 ➔ $I_{yc,bf} / I_y = 0.09 \le 0.23$ ➔ $R_{pt} = 1.00$

• **Lateral-torsional buckling** $M_{nc,pos4} = F_{cr} S_{x,bf}$ **= 884 kN ·m**

- The effective radius of gyration $r_t = \dfrac{b_{fc}}{\sqrt{12\left(1 + a_w / 6\right)}} = 101.51$ mm

  where, width of compression flange $b_{fc} = b_p = 382$ mm and $a_w = \dfrac{2\left(C_y + c_o\right)t_w}{b_{fc} t_p} = 1.08$

- The limiting laterally unbraced length for the limit state of yielding $L_p = 1.1 r_t \sqrt{\dfrac{E}{F_y}}$ = **2715** mm
- The limiting unbraced length for the limit state of inelastic lateral-torsional buckling

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{x,bf}h_o} + \sqrt{\left(\frac{J}{S_{x,bf}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}} = \mathbf{9726}\ \text{mm}$$

L_b = **10000** mm ➔ $L_r < L_b$

where,

Torsional constant for thin walled open tube $J = \dfrac{1}{3}\left[2\left(H - t_f\right)t_w^{\,3} + 2b_f t_f^{\,3} + b_p t_p^{\,3}\right]$ = 565578 mm^4

$h_o = H - t_f/2 - t_p/2 = 574.5$ mm

3. Calculation sheet (TG1)

2. Check Construction Stage > 2.4 Shear Strength

- **Required shear strength** $V_c = [(1.2W_d + 1.6W_c)\times B_{ay} + 1.2W_s]\times L/2$ = **184.67 kN**
- For webs without transverse stiffeners, the web plate shear buckling coefficient $k_v = 5.34$
- The web shear strength coefficient, C_v, is determined as follows:

$$h/t_w = 591/9 = 65.66 > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{5.34\times 210000/355} = 61.82 \rightarrow C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} = 0.94$$

- **The nominal shear strength** $\Phi_v V_n = 0.9\times 0.6F_y(2t_w h)C_v = 0.9\times 0.6\times 355\times 10638\times 0.96 =$ **1957.73 kN > 184.67 kN … O.K.**

AISC 360-16 > G. Design of members for shear > **G2. I-shaped members > 1. Shear strength of webs without tension filed action**

2. Check Construction Stage > 2.5 Deflection

- **Deflection :** $\delta_{const} = \frac{W_d\times B_{ay} + W_s}{384E_s I_x}\times L^4$ **= 3.1 mm < *L*/360 = 27.7 mm or 25 mm … O.K.**
- Boundary condition : Fix-Fix
- The self-weight of concrete is considered as the applied load during concrete curing

AISC 360-16 > I. Design of composite members > I3. Flexure > 1b. Strength during construction

When temporary shores are not used during construction, the steel section alone shall have sufficient strength to support all loads prior to the concrete attaining 75% of its specified strength, f'_c.

(comment) Composite beam design requires care in considering the loading history. Loads applied to an unshored beam before the concrete has cured are resisted by the steel section alone; total loads applied before and after the concrete has cured are considered to be resisted by the composite section.

3. Calculation sheet (TG1)

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3. Check Composite Section > 3.1 Width-Thickness Ratio

- **Top flange** : $(b_{f,out} - t_w/2)/\ t_f = 91\ /\ 9 = 9.6$
- **Web** : $(h\text{-}c_o\text{-}t_p)/t_w = 62.6 < \lambda_p = 3.00\sqrt{E_s\ /\ F_y} = 73.0$ mm ➔ **Compact section**
- **Bottom flange** : $b_p/t_p = 31.8 < \lambda_p = 2.26\sqrt{E_s\ /\ F_y} = 57.6$ mm ➔ **Compact section**

Table I1.1b

AISC 360-16 > I. Design of composite members > I1.4. Classification of **filled composite sections** for local buckling

Table I1.1b Limiting width-to-thickness ratios for compression steel elements in composite members subject to flexure

3. Check Composite Section > 3.2 Composite Ratio

- Effective width of concrete slab $B_{eff} = \min[B_{ay}, L/4] = \min[3000, 2500] = 2500$ mm

AISC 360-16 > I. Design of composite members > I3. Flexure > 1a. Effective width

The effective width of the concrete slab shall be the sum of the effective widths for each side of the beam centerlines, each of which shall not exceed:

(a) one-eighth of the beam span, center-to-center of supports;

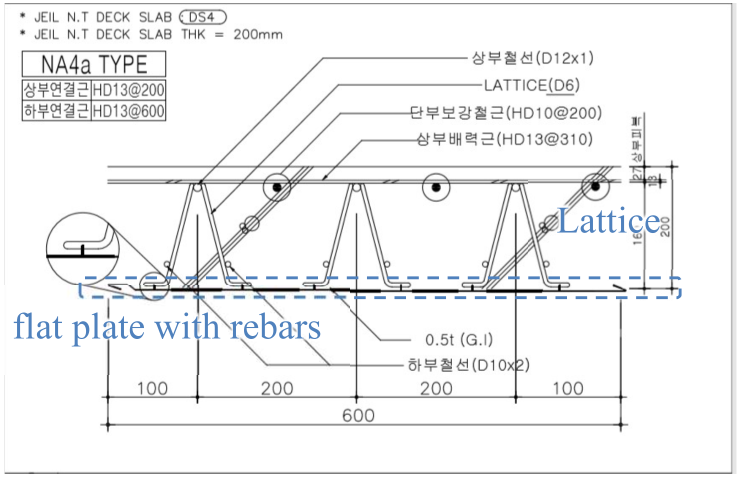
(b) one-half the distance to the centerline of the adjacent beam; or

(c) the distance to the edge of the slab.

(comment) The same effective width ruled apply to composite beams with a slab on either one side or both sides of the beam. ~ To simplify design, the effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

3. Calculation sheet (TG1)

3. Check Composite Section > 3.2 Composite Ratio



(1) Shear connector strength

- **The nominal shear strength of one steel headed stud anchor embedded in a solid concrete slab Q_n**

$$Q_n = 0.5 A_{sa} \sqrt{f_{ck} E_c} \leq R_g R_p A_{sa} F_u \quad = \mathbf{85.06\ kN}$$

where

A_{sa} = cross-sectional area of steel headed stud anchor (= 283 mm^2)

R_g = 1.00 for any number of steel headed stud anchors welded in a row directly to the steel shape

R_p = 0.75 for steel headed stud anchors welded directly to the steel shape

Decking with flat plate

F_u = Specified minimum tensile strength of a steel headed stud anchor (= 400 MPa)

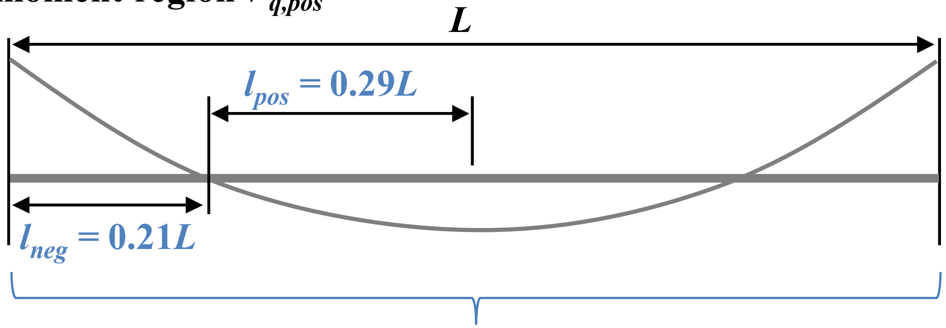
AISC 360-16 > I. Design of composite members > I3. Flexure > 1a. Effective width

(comment) The reduction factor, R_p, for headed stud anchors used in ***composite beams with no decking was reduced from 1.0 to 0.75*** *in the 2010 AISC specification. The research (Roddenberry et al., 2002a) in which the factors R_g and R_p were developed focused almost exclusively on cases involving the use of headed stud anchors welded through the steel deck. The research pointed to the likelihood that the slab case should use R_p = 0.75; however, the body of test data had not been established to support the change. More recent research has shown that the 0.75 factor is appropriate (Palleres and Hajjar, 2010a)*

3. Calculation sheet (TG1)

3. Check Composite Section > 3.2 Composite Ratio

(2) Sum of nominal shear strengths of steel headed stud under positive moment region $V_{q,pos}$



Shear connectors: 2Row-Φ19@200

- $l_{pos} = 0.29L = 2900$ mm
- The number of shear connectors $n_{s,p} = 2\times2900/200 = 30$
- $V_{q,pos} = \sum Q_n = Q_n n_s =$ **2551.8 kN**

(3) Sum of nominal shear strengths of steel headed stud under negative moment region $V_{q,neg}$

- $l_{neg} = 0.21L = 2100$ mm
- The number of shear connectors $n_{s,n} = 2\times2100/200 = 22$
- $V_{q,neg} = \sum Q_n = Q_n n_s =$ **1871.32 kN**

3. Check Composite Section > 3.3 Flexure Strength

(1) Design strengths

- The required positive moment strength $M_{u,pos} = 712$ kN·m
- The required negative moment strength $M_{u,neg} = 1425$ kN·m
- (Dead load) Superimposed load $W_f = 5550$ N/m^2
- Live load $W_l = 15000$ N/m^2

3. Calculation sheet (TG1)

3. Check Composite Section > 3.3 Flexure Strength

(2) Positive moment

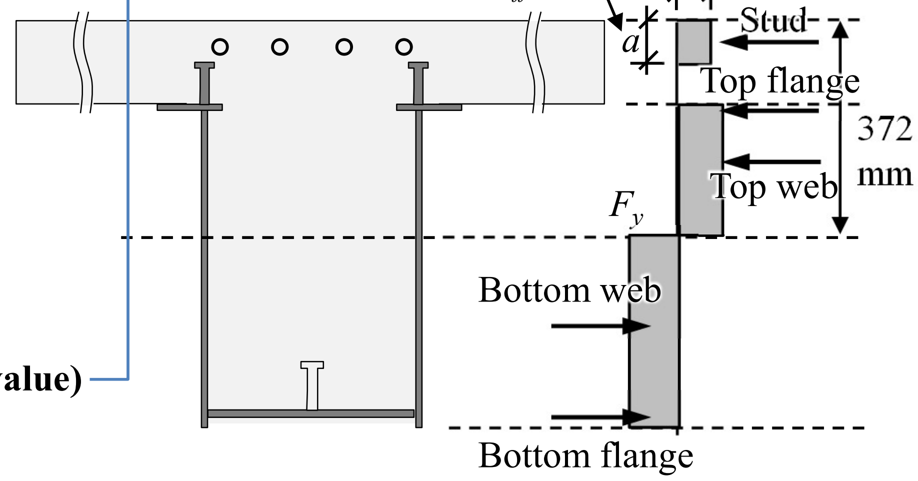
Load transfer between steel beam and concrete slab

- Concrete crushing $V_{slab} = 0.85 f_{ck} A_c = 0.85 \times 30 \times 2500 \times d_a = 6375$ kN

  where, d_a = depth of PNA with fully composite action
- Tensile yielding of the steel section $V_{steel} = F_y A_s = 355 \times 17982 = 6383$ kN
- **Shear strength of steel headed stud $V_{stud} = V_{q,pos} = 2551$ kN (minimum value)**
- $a = 40$ mm; Plastic neutral axis $C_{y,pc} = 390$ mm
- The compressive resistance by longitudinal slab reinforcements and concrete infill are neglected.
- **The nominal plastic moment strength in positive bending $M_{n,pos} = 2386$ kN·m**

  ➔ $\Phi M_{n,pos} = 0.9 \times 2386 = 2147$ kN·m > $M_{u,pos} = 712$ kN·m … O.K.

$$a = \frac{V_{stud}}{0.85 f_{ck} B_{eff}}$$



AISC 360-16 > I. Design of composite members > I3. Flexure > 2d. Load transfer between steel beam and concrete slab

(1. Load transfer for positive flexural strength) <u>The entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by steal headed stud</u>. For composite action with concrete subject to flexural compression, the nominal shear force between the steel beam and the concrete slab transferred by steel anchors, *V'*, between the point of maximum positive moment and the point of zero moment of concrete crushing, tensile yielding of the steel section, or the shear strength of the steel anchors.

(comment) The flexural strength of a composite beam in the positive moment region may be controlled by the strength of the steel section, the concrete slab, or the steel headed anchors. ~ Longitudinal slab reinforcement makes a negligible contribution to the compression force,

3. Calculation sheet (TG1)

3. Check Composite Section > 3.3 Flexure Strength

(2) Negative moment

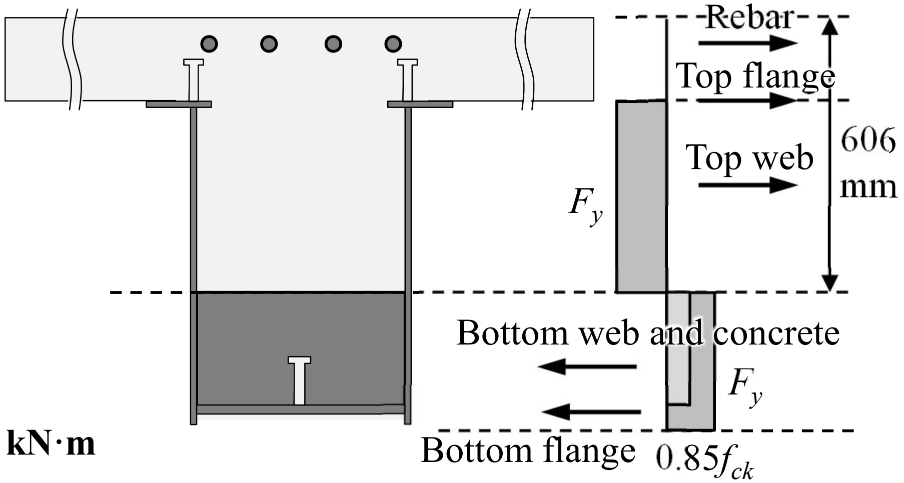
Load transfer between steel beam and concrete slab

- **Tensile yielding of the slab reinforcement**

  $V_{sr} = F_{yr}A_{sr}$ **= 600 × 1962 = 1177 kN (minimum value)**

- Shear strength of steel headed stud $V_{stud} = V_{q,neg} = 1870$ kN
- Plastic neutral axis $C_{y,pl} = 623$ mm
- **The nominal plastic moment strength in negative bending** $M_{n,neg}$ **= 2015 kN·m**

  ➔ $\Phi M_{n,neg}$ **= 0.9×2015 = 1813 kN·m > $M_{u,neg}$ = 1425 kN·m … O.K.**



AISC 360-16 > I. Design of composite members > I3. Flexure > 2d. Load transfer between steel beam and concrete slab

(1. Load transfer for positive flexural strength) In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear between the point of the maximum negative moment and the point of zero moment shall be determined as the lower value in accordance with the following limit states.

(comment) When and adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distribution. Loads applied to a continuous composite beam with steel anchors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

3. Calculation sheet (TG1)

3. Check Composite Section > 3.4 Shear Strength

- The required shear strength $V_u = 610$ kN
- The nominal shear strength $\Phi V_n = 0.9 \times 0.6F_y\,(2ht_w) = 0.9\times0.6\times355\times10638 = 2039$ kN $> V_u = 610$ kN **… O.K.**

  where, A_w = the overall depth times the web thickness, (mm^2)

AISC 360-16 > I. Design of composite members > I4. Shear

(1. Filled and encased composite members) The design shear strength, $\Phi_v V_n$, shall be determined based on one of the following:

(a) The available <u>shear strength of the steel section alone</u> as specified in Chapter G.

(comment) ~ Though it would be logical to suggest provisions where both the contributions of the steel section and reinforced concrete are superimposed, there is insufficient research available to justify such a combination.

(2. Composite beams with formed steel deck) The available shear strength of composite beams with steel headed stud or steel channel anchors shall be determined based upon <u>the properties of the steel section alone</u> in accordance with chapter G.

(comment) <u>A conservative approach</u> to shear provisions for composite beams with steel headed stud or steel channel anchors <u>is adopted by assigning all shear to the steel section</u> in accordance with Chapter G. <u>This method neglects any concrete contribution and serves to simplify design</u>.

3. Calculation sheet (TG1)

3. Check Composite Section > 3.5 Deflection (short-term)

- Boundary condition : Fix-Fix
- The equivalent moment of inertia, $I_{equiv} = I_s + \sqrt{\left(\sum Q_n / C_f\right)}\left(I_{tr} - I_s\right)$ = 335197 × 10^4 mm^4

  where,

  I_s = moment of inertia for the structural section (= I_x = 82808 × 10^4 mm^4)

  $\sum Q_n$ = sum of the nominal strengths of steel anchors between the point of maximum positive moment and the point of zero moment to either side (= $V_{q,pos}$ = 2550 kN)

  C_f = compression force in concrete slab for fully composite beam (= tensile yielding of the steel section $V_{steel} = F_y A_s$ = 6170 kN)

  I_{tr} = moment of inertia for the fully composite uncracked transformed section (= 475402 mm^4; the ratio of E_c/E_s is multiplied by the term elastic modulus of concrete)

  - In the term I_{equiv}, boundary condition is assumed as a pin-pin condition for a conservative estimation.

AISC 360-16 > I. Design of composite members > I3. Flexure > 2. Composite beams with steel headed stud

(comment) When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. ~ More recent studies indicate that the use of the equivalent moment of inertia, I_{equiv}, for deflection calculation results in a prediction of short-term deflections roughly equivalent to the statistical average average of the experimental tests reviewed. Previous editions of the Specification recommended an additional reduction factor 0.75 be applied to I_{equiv} to form an effective moment of inertia; however, this approach has been removed as its basis could not be substantiated.

3. Calculation sheet (TG1)

3. Check Composite Section > 3.5 Deflection (short-term)

- **Deflection** $\delta_{add} = \dfrac{(W_f + W_l) \times B_{ay}}{384 E_s I_{equiv}} \times L^4$ **= 2.3 mm**

  where,

  (Dead load) Superimposed load W_f = 5550 N/m^2

  Live load W_l = 15000 N/m^2

- **Total deflection δ_{comp}** with dead and live loads $W_d + W_s + W_f + W_l$,

  $\delta_{st} = \delta_{const} + \delta_{add}$ **= 3.1 + 2.3 = 5.4 mm < L/ 360 = 27.7 mm** **… O.K.**

Deflection δ_{const} during concrete curing (resisted by steel section)

⇩

Additional deflection δ_{add} after concrete curing (resisted by composite section)

⇩

Total deflection $\delta_{comp} = \delta_{const} + \delta_{add}$

AISC 360-16 > L. Design for serviceability > L2. Deflections

Deflections in structural members and structural systems shall be limited so as not to impair the serviceability of the structure.

(comment) ~ Historically, common deflection limits for horizontal members have been ***1/360 of the span for floors subjected to reduced live load*** *and 1/240 of the span for roof members.* ***Deflection of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage****. Deflection greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions. ~ Load combinations for checking static deflections can be developed using first-order reliability analysis. Current static deflection guidelines for floor and roof systems are adequate for limiting superficial damage in most buildings. A combined load with an annual probability of being exceeded of 5% is appropriate in most instances. For* ***serviceability limit states*** *involving visually objectionable deformations, repairable cracking, or other damage to interior finishes, and* ***other short-term effects****, the suggested load combinations are:*

$$D + L$$

3. Calculation sheet (TG1)

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3. Check Composite Section > 3.5 Deflection (short-term)

IBC2015 > 16. Structural design > Section 1604 General design requirements > Table 1604.3 Deflection limits

CONSTRUCTION	L	S or W^{f}	$D + L^{d,g}$
Roof members:[e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240

d. **The deflection limit for the *D* + *L* load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection**.

3. Check Composite Section > 3.5 Deflection (long-term)

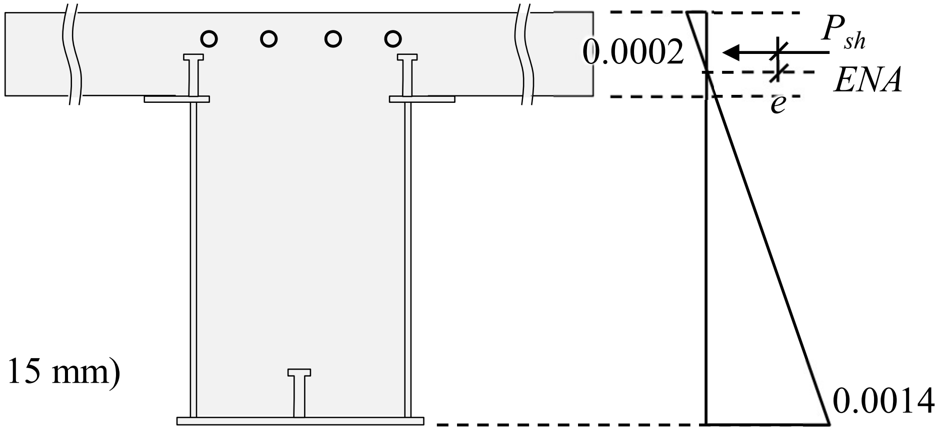
- The shrinkage strain, e_{sh}, may be taken as 0.0002.
- The shrinkage load $P_{sh} = e_{sh}E_cA_c = 0.0002 \times 27537 \times 600000 = 3304$ kN

AISC 360-16 > I. Design of composite members > I3. Flexure > 2. Composite beams with steel headed stud

(comment) ***Long-term deformations due to shrinkage and creep***: *There is no direct guidance in the computation of long-tern deformations of composite beams due to creep and shrinkage. The long-term deformation due to shrinkage can be calculated with the simplified model, in which the effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained shrinkage strain times modulus of concrete times effective area of concrete) times the eccentricity between the center of the slab and the elastic neutral axis. ~*

3. Calculation sheet (TG1)

3. Check Composite Section > 3.5 Deflection (long-term)



- The shrinkage strain, e_{sh}, may be taken as 0.0002.
- The shrinkage force $P_{sh} = e_{sh}E_cA_c = 0.0002 \times 27537 \times 600000 = 3304$ kN
- **The deflection due to shrinkage** $\delta_{lt} = \dfrac{P_{sh}eL^2}{8E_sI_x}$ **= 3.6 mm**

  where,

  e = eccentricity between the center of the slab and the elastic neutral axis (= 15 mm)

  I_x = moment of inertia of steel section (=82808×10^4 mm^4)
- $\boldsymbol{\delta_{st} + \delta_{lt}}$ **= 9.0 mm < $\boldsymbol{L}$/240 = 41 mm … O.K.**