3. Calculation sheet (TG1)

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/m²
- Construction load $W_c = 2500$ N/m²; 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

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)

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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,

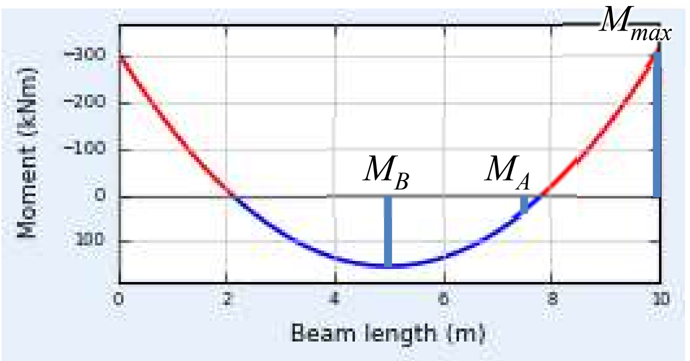
Torsional constant for thin walled open tube $J = \frac{1}{3}\left[2\left(H - t_f\right)t_w^{\,3} + 2b_f t_f^{\,3} + b_p t_p^{\,3}\right]$ = 565578 mm^4

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

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



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 $I_{yc,bf} = 5574 \times 10^4$ mm^4

  - Moment of inertia about the principal axis $I_y = 57580 \times 10^4$ 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 $M_{yc} = F_y S_{x,bf} = 355 \times 3380 \times 10^3 = 1199.9\ \text{kN·m}$

- $\dfrac{S_{x,tf}}{S_{x,bf}} = \dfrac{2430 \times 10^3}{3380 \times 10^3} = 0.72 > 0.7$ ➔ $F_L = 0.7 F_y = 248.50\ \text{MPa}$

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 $I_{yc,bf} = 5574 \times 10^4\ \text{mm}^4$

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

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

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

  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 $J = \dfrac{1}{3}\left[2\left(H - t_f\right)t_w^{\ 3} + 2 b_f t_f^{\ 3} + b_p t_p^{\ 3}\right] = 565578\ \text{mm}^4$

$h_o = H - t_f/2 - t_p/2 = 574.5$ 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** $\Phi_v V_n = 0.9\times 0.6 F_y (2t_w h) C_v = 0.9\times 0.6\times 355\times 10638\times 0.96 =$ **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 :** $\delta_{const} = \frac{W_d\times B_{ay} + W_s}{384 E_s I_x}\times L^4$ **= 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;

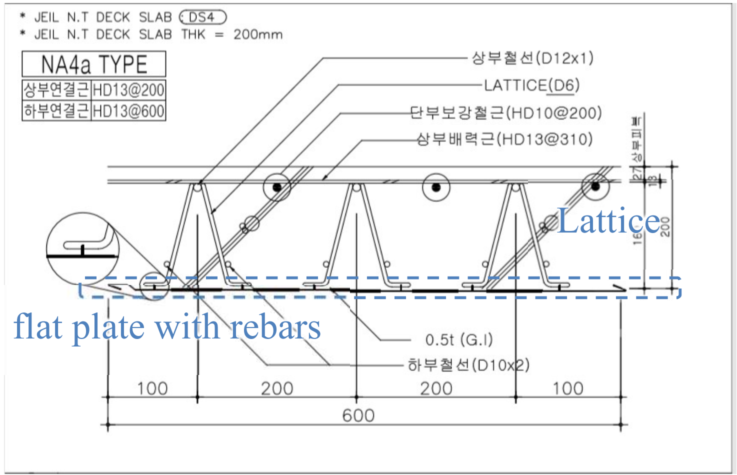
(b) one-half the distance to the centerline of the adjacent beam; or

(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**

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

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)

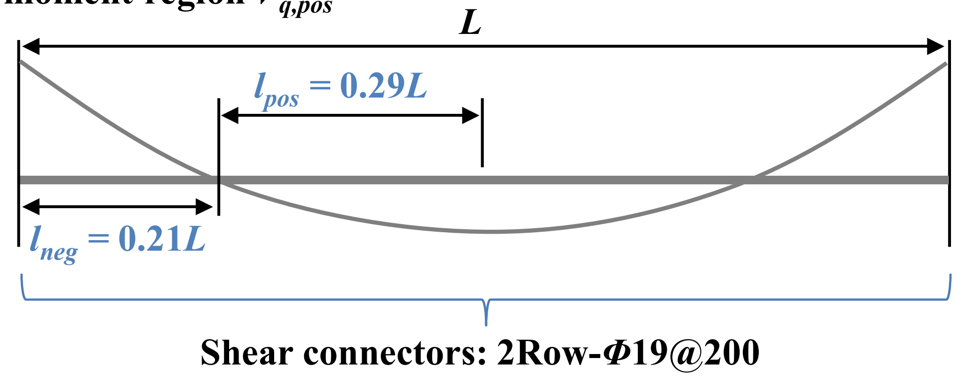
AISC 360-16 > I. Design of composite members > I3. Flexure > 1a. Effective width

(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 $V_{q,pos}$



- $l_{pos} = 0.29L = 2900$ 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

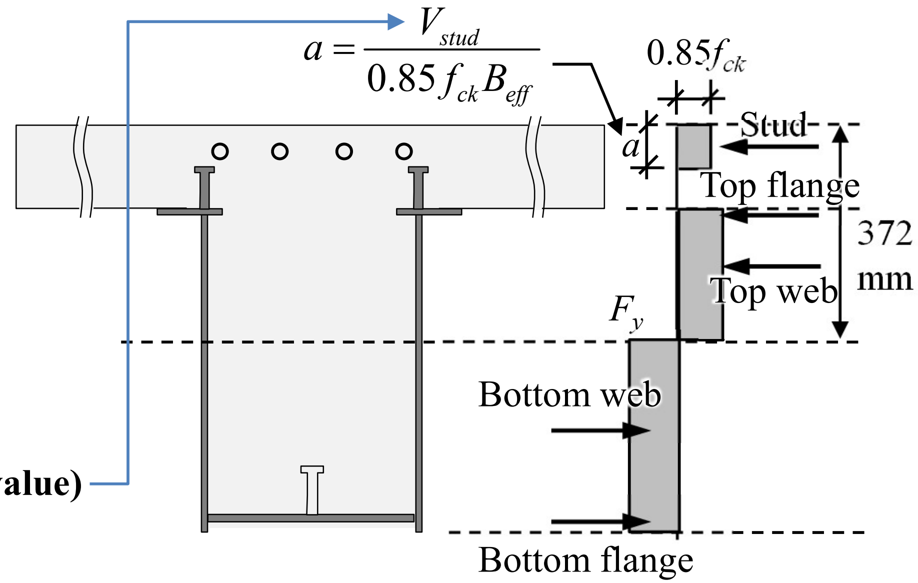
(2) Positive moment

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.



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,