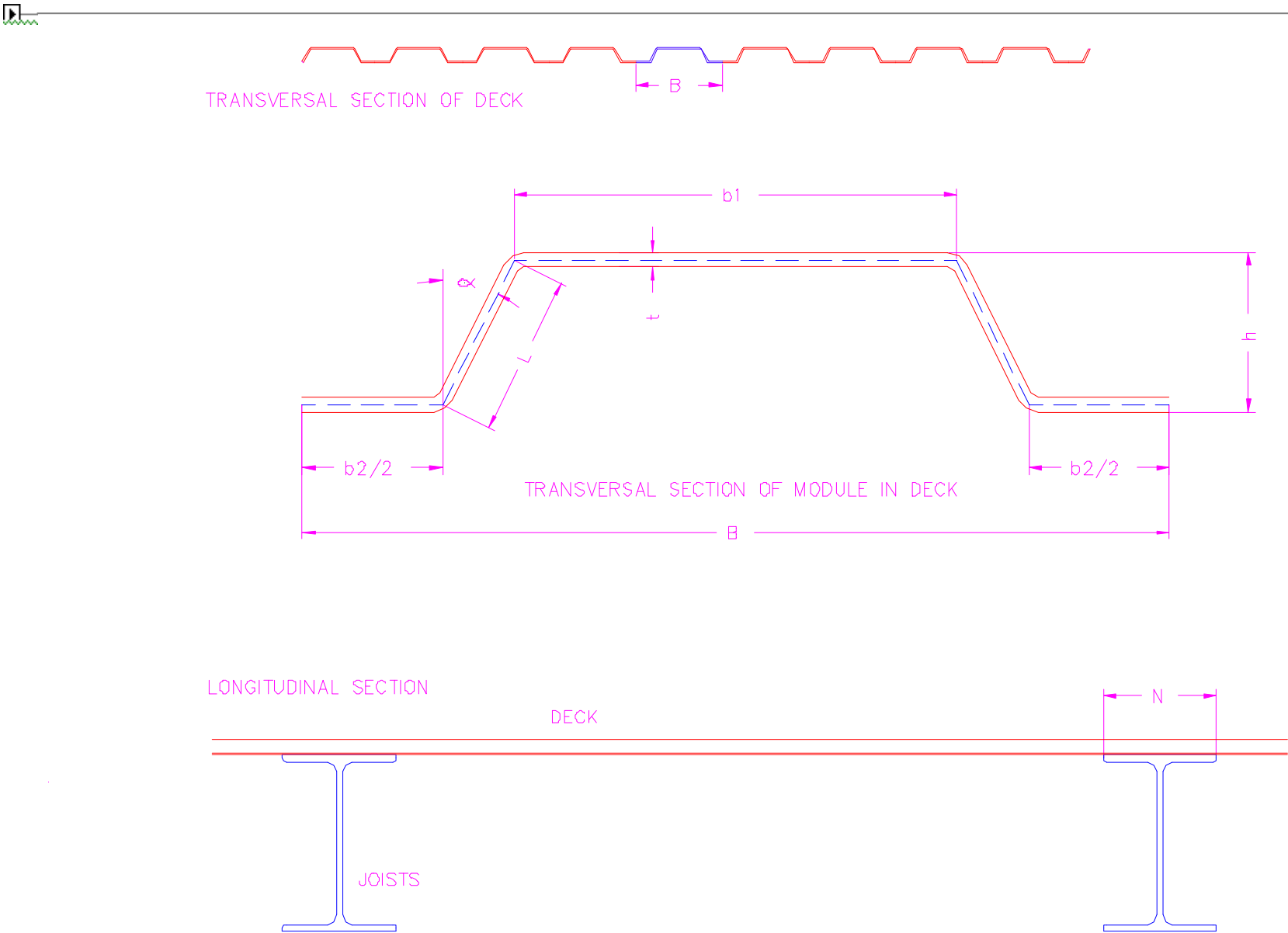


Strength of a Sheet Metal Trapecial Deck Section

Release 3



$F_y := 260 \cdot \text{MPa}$

$E := 200000 \cdot \text{MPa}$

$\nu := 0.3$

of steel in deck

$B := 30 \cdot \text{cm}$

$b_1 := 15 \cdot \text{cm}$

width flat atop, interaxial

$t := 1.2 \cdot \text{mm}$

steel thickness

width of module

$b_2 := 8 \cdot \text{cm}$

width flat bottom, interaxial

$h := 5 \cdot \text{cm}$

measured vertically between
bottom and top faces of flanges

Factored forces at the section per B width module...

$$M_u := -0.08 \cdot \text{m} \cdot \text{ton}$$

- positive if compresses atop
- negative if compresses at bottom

$$\phi_b := 0.9$$

$$V_u := 0.8 \cdot \text{ton}$$

concurrent shear at the same station than Mu

$$\phi_v := 0.9$$

Moment-Shear interaction



The compressed flange effective width will depend on what average stress corresponds there, which may be less than F_y if the tension flange is big enough (and so giving a bigger effective width of the compression flange).

$$\alpha := \text{atan}\left(\frac{\frac{B-b_1-b_2}{2}}{h-t}\right) \quad \alpha = 35.65 \text{ deg}$$

$$L_{\text{eff}} := \frac{h}{\cos(\alpha)} \quad L = 6.15 \text{ cm}$$

$$k := 4 \quad \text{flanges are assumed fully supported at both sides by webs}$$

Forfeit any worry of sectional deformation due to lack of lateral support at lateral ends of decks. Normally they will be welded to support or fastened to adjacent deck panel. Even if not, interlock and sectional (frame-like) rigidity will allow so little for sectional deformation that any significant structural effect may be discounted with normal loads and decks.

Top flange in compression at limit strength

$$\sigma_{e1} := F_y \quad \text{unwarranted guess for applied stress (on effective width) atop when the section attains the moment strength with compression atop}$$

$$\sigma_{c1} := k \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_1}{t}\right)^2} \quad \text{critical elastic buckling strength of the uniformly compressed flange}$$

$$b_{e1}(\sigma_{e1}) := b_1 \cdot \min \left[\left[\frac{1}{\sqrt{\frac{\sigma_{c1}}{\sigma_{e1}}}} \cdot \left(1 - 0.22 \cdot \sqrt{\frac{\sigma_{c1}}{\sigma_{e1}}} \right) \right] \right]$$

AISI formulation for effective width at the stress level, on such maximum applied stress anywhere (and so to be ported upon this effective width)

Now we can determine the effective modulus of section for the being tested applied stress on the effective width.

$$y_{g_tc}(\sigma_{e1}) := \frac{b_{e1}(\sigma_{e1}) \cdot t \cdot \left(h - \frac{t}{2} \right) + 2 \cdot L \cdot t \cdot \frac{h}{2} + b_2 \cdot t \cdot \frac{t}{2}}{b_{e1}(\sigma_{e1}) \cdot t + 2 \cdot L \cdot t + b_2 \cdot t}$$

$$I_{tc}(\sigma_{e1}) := b_{e1}(\sigma_{e1}) \cdot t \cdot \left(h - \frac{t}{2} - y_{g_tc}(\sigma_{e1}) \right)^2 + b_2 \cdot t \cdot \left(y_{g_tc}(\sigma_{e1}) - \frac{t}{2} \right)^2 + 2 \cdot \frac{t}{\cos(\alpha)} \cdot \frac{h^3}{12}$$

it is assumed the web will remain fully effective, what will happen for normal deck designs

$$S_{1_tc}(\sigma_{e1}) := \frac{I_{tc}(\sigma_{e1})}{\left(h - y_{g_tc}(\sigma_{e1}) \right)} \quad \text{modulus of section for top fiber}$$

$$S_{2_tc}(\sigma_{e1}) := \frac{I_{tc}(\sigma_{e1})}{y_{g_tc}(\sigma_{e1})} \quad \text{modulus of section for bottom fiber}$$

We won't allow any plastic strength for the thin section.
The flange with lesser modulus of section will attain sooner the controlling limit stress F_y , hence

$$M_{n_tc}(\sigma_{e1}) := F_y \cdot \min \left(\left(S_{1_tc}(\sigma_{e1}) \right), \left(S_{2_tc}(\sigma_{e1}) \right) \right)$$

Given

$$\sigma_{e1} = \frac{M_{n_tc}(\sigma_{e1})}{S_{1_tc}(\sigma_{e1})}$$

since in any case and by definition the testing applied stress on effective width must correspond to that at the limit momet strength with compression atop

$$\sigma_{e1} := \text{Maximize}(M_{n_tc}, \sigma_{e1})$$

$$M_{n_tc}(\sigma_{e1}) = 0.12 \text{ m}\cdot\text{ton}$$

limit moment strength (with compression atop) for the module

$$\frac{M_{n_tc}(\sigma_{e1})}{B} = 0.41 \frac{\text{m}\cdot\text{ton}}{\text{m}}$$

limit moment strength (with compression atop) per unit width of deck

$$\sigma_{e1} = 1 F_y$$

stress at top fiber at limit moment strength compressing atop, equals

$$\frac{M_{n_tc}(\sigma_{e1})}{S_{1_tc}(\sigma_{e1})} = 1 F_y$$

$$b_{e1}(\sigma_{e1}) = 0.38 b_1$$

$$b_{e1}(\sigma_{e1}) = 38.27 \% \cdot b_1$$

the width of top flange that is effective at the maximum moment strength with compression in the top flange

$$\frac{M_{n_tc}(\sigma_{e1})}{S_{2_tc}(\sigma_{e1})} = 0.84 F_y$$

stress at bottom fiber at limit moment strength compressing atop

Bottom flange in compression at limit strength

$$\sigma_{e2} := F_y$$

unwarranted guess for applied stress (on effective width) atop when the section attains the moment strength with compression atop

$$\sigma_{c2} := k \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \nu^2) \cdot \left(\frac{b_2}{t}\right)^2}$$

critical elastic buckling strength of the uniformly compressed flange

$$b_{e2}(\sigma_{e2}) := b_2 \cdot \min \left[\left[\frac{1}{\sqrt{\frac{\sigma_{c2}}{\sigma_{e2}}}} \cdot \left(1 - 0.22 \cdot \sqrt{\frac{\sigma_{c2}}{\sigma_{e2}}} \right) \right] \right]$$

AISI formulation for effective width at the stress level, on such maximum applied stress anywhere (and so to be ported upon this effective width)

Now we can determine the effective modulus of section for the being tested applied stress on the effective width.

$$y_{g_bc}(\sigma_{e2}) := \frac{b_1 \cdot t \cdot \left(h - \frac{t}{2}\right) + 2 \cdot L \cdot t \cdot \frac{h}{2} + b_{e2}(\sigma_{e2}) \cdot t \cdot \frac{t}{2}}{b_1 \cdot t + 2 \cdot L \cdot t + b_{e2}(\sigma_{e2}) \cdot t}$$

$$I_{bc}(\sigma_{e2}) := b_1 \cdot t \cdot \left(h - \frac{t}{2} - y_{g_bc}(\sigma_{e2})\right)^2 + b_{e2}(\sigma_{e2}) \cdot t \cdot \left(y_{g_bc}(\sigma_{e2}) - \frac{t}{2}\right)^2 + 2 \cdot \frac{t}{\cos(\alpha)} \cdot \frac{h^3}{12}$$

it is assumed the web will remain fully effective, what will happen for normal designs

$$S_{1_bc}(\sigma_{e2}) := \frac{I_{bc}(\sigma_{e2})}{\left(h - y_{g_bc}(\sigma_{e2})\right)}$$

modulus of section for top fiber

$$S_{2_bc}(\sigma_{e2}) := \frac{I_{bc}(\sigma_{e2})}{y_{g_bc}(\sigma_{e2})}$$

modulus of section for bottom fiber

We won't allow any plastic strength for the thin section.

The flange with lesser modulus of section will attain sooner the controlling limit stress F_y , hence

$$M_{n_bc}(\sigma_{e2}) := F_y \cdot \min \left(\left(\frac{S_{1_bc}(\sigma_{e2})}{S_{2_bc}(\sigma_{e2})} \right) \right)$$

Given

$$\sigma_{e2} = \frac{M_{n_bc}(\sigma_{e2})}{S_{2_bc}(\sigma_{e2})}$$

since in any case and by definition the testing applied stress on effective width must correspond to that at the limit momet strength with compression bottom

$$\sigma_{e2} := \text{Maximize}(M_{n_bc}, \sigma_{e2})$$

$$M_{n_bc}(\sigma_{e2}) = 0.12 \text{ m}\cdot\text{ton}$$

limit moment strength (with compression at bottom) for the module

$$\frac{M_{n_bc}(\sigma_{e2})}{B} = 0.4 \frac{\text{m}\cdot\text{ton}}{\text{m}}$$

limit moment strength (with compression at bottom) per unit width of deck

$$\frac{M_{n_bc}(\sigma_{e2})}{S_{1_bc}(\sigma_{e2})} = 0.55 F_y$$

stress at top fiber at limit moment strength compressing at bottom

$$\sigma_{e2} = 1 F_y$$

stress at bottom fiber at limit moment strength compressing bottom, equals

$$\frac{M_{n_bc}(\sigma_{e2})}{S_{2_bc}(\sigma_{e2})} = 1 F_y$$

$$b_{e2}(\sigma_{e2}) = 0.65 b_2$$

$$b_{e2}(\sigma_{e2}) = 65.34 \% \cdot b_2$$

the width of bottom flange that is effective at the maximum moment strength with compression in the bottom flange

Use these to balance your design for your needs

$M_{n_bc}(\sigma_{e2}) = 96.76\% \cdot M_{n_tc}(\sigma_{e1})$

negative moment capacity as percent of positive moment capacity

$M_{n_tc}(\sigma_{e1}) = 103.35\% \cdot M_{n_bc}(\sigma_{e2})$

positive moment capacity as percent of negative moment capacity

Shear strength of the unstiffened web

$F_{yw} := F_y$

$k_v := 5$

for any unstiffened web

$\lambda_w := \frac{L}{t}$

$F_{v_cr}(\lambda_w) :=$

$0.6 \cdot F_{yw}$

if

$\lambda_w \leq 187 \cdot \sqrt{\frac{k_v}{\frac{F_{yw}}{\text{ksi}}}}$

otherwise

$0.6 \cdot F_{yw} \cdot \frac{187 \cdot \sqrt{\frac{k_v}{\frac{F_{yw}}{\text{ksi}}}}}{\lambda_w}$

if

$187 \cdot \sqrt{\frac{k_v}{\frac{F_{yw}}{\text{ksi}}}} < \lambda_w \leq \frac{187}{0.8} \cdot \sqrt{\frac{k_v}{\frac{F_{yw}}{\text{ksi}}}}$

otherwise

$k_v \cdot \frac{\pi^2 \cdot E}{12 \cdot (1 - \nu^2) \cdot \lambda_w^2}$

$V_n := 2 \cdot L \cdot t \cdot F_{v_cr}(\lambda_w) \cdot \cos(\alpha)$

$V_n = 1.91 \text{ ton}$

limit shear (vertical)

$M_n := \begin{cases} M_{n_tc}(\sigma_{e1}) & \text{if } M_u > 0 \cdot \text{m} \cdot \text{ton} \\ M_{n_bc}(\sigma_{e2}) & \text{otherwise} \end{cases}$

$I_{eff} := \begin{cases} I_{tc}(\sigma_{e1}) & \text{if } M_u > 0 \cdot \text{m} \cdot \text{ton} \\ I_{bc}(\sigma_{e2}) & \text{otherwise} \end{cases}$

$Ratio_{MV} := \left(\frac{M_u}{\phi_b \cdot M_n} \right)^2 + \left(\frac{V_u}{\phi_v \cdot V_n} \right)^2$



$Ratio_{MV} = 0.77$

Must be equal to or less than 1 for the section be OK under the combined bending plus shear interaction

$I_{eff} = 14.64 \text{ cm}^4$

Effective moment of inertia at the section, just in case you want to feed it back to the analysis

Web Crippling

$P_u := 1.6 \cdot \text{ton}$

factored reaction at support, per module of width B

$N_{\text{www}} := 8 \cdot \text{cm}$

width of bearing at support, along span of deck



$t_w := t$

$t_f := t$

$d_p := 0 \cdot \text{cm}$

since reaction

$\text{h}_{\text{www}} := L$

$\phi := 0.75$

$$R_n := \cos(\alpha) \cdot 2 \cdot \text{ksi} \cdot \left\{ \begin{array}{l} 135 \cdot t_w^2 \cdot \left[1 + 3 \cdot \frac{N}{h} \cdot \left(\frac{t_w}{t_f} \right)^{1.5} \right] \cdot \sqrt{\frac{F_{yw}}{\text{ksi}} \cdot \frac{t_f}{t_w}} \quad \text{if } d_p \geq \frac{h}{2} \\ \text{otherwise} \\ \left\{ \begin{array}{l} 68 \cdot t_w^2 \cdot \left[1 + 3 \cdot \frac{N}{h} \cdot \left(\frac{t_w}{t_f} \right)^{1.5} \right] \cdot \sqrt{\frac{F_{yw}}{\text{ksi}} \cdot \frac{t_f}{t_w}} \quad \text{if } \frac{N}{h} \leq 0.2 \\ 68 \cdot t_w^2 \cdot \left[1 + \left(4 \cdot \frac{N}{h} - 0.2 \right) \cdot \left(\frac{t_w}{t_f} \right)^{1.5} \right] \cdot \sqrt{\frac{F_{yw}}{\text{ksi}} \cdot \frac{t_f}{t_w}} \quad \text{otherwise} \end{array} \right. \end{array} \right.$$

$$\text{Stiffening} := \left\{ \begin{array}{l} \text{"Required"} \quad \text{if } P_u > \phi \cdot R_n \\ \text{"NOT required"} \quad \text{otherwise} \end{array} \right.$$



Stiffening = "NOT required"

If stiffening is required you must select a thicker sheet metal deck, or modify the layout till not needed