CSC TEDDS	Project			Job Ref.	
CSC Inc 500 N Michigan Ave Chicago, IL 60611	LRFD Girder Design Example			Design Examples	
	Section			Sheet no./rev.	
	Composite Beam Design			1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	JRE	4/7/2009	AJR	4/8/2009	JRE	4/9/2009

FASTRAK Composite Beam Design

LRFD Girder Design Example Calculation

FASTRAK Composite Beam Design is a design tool for composite and non-composite beams with flexible loading options, design criteria, and stud optimization and placement. This powerful tool is available **FREE** in the US and can be downloaded from http://www.cscworld.com/fastrak/us/composite_download.html

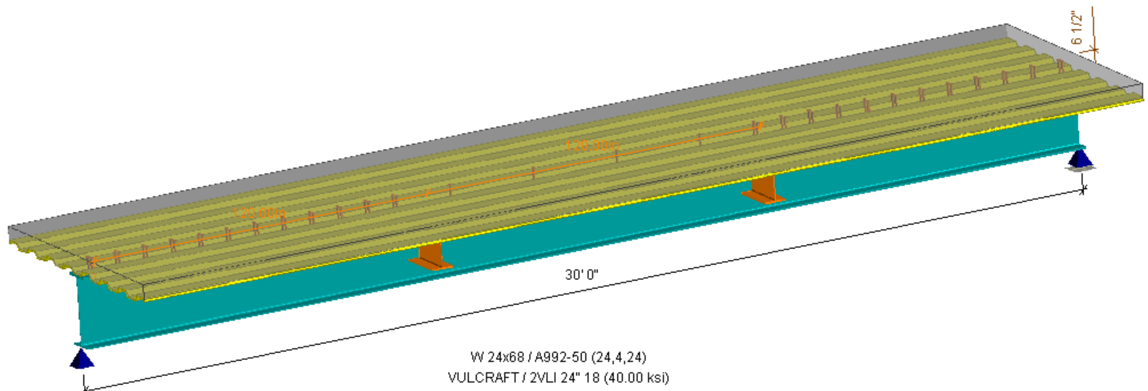
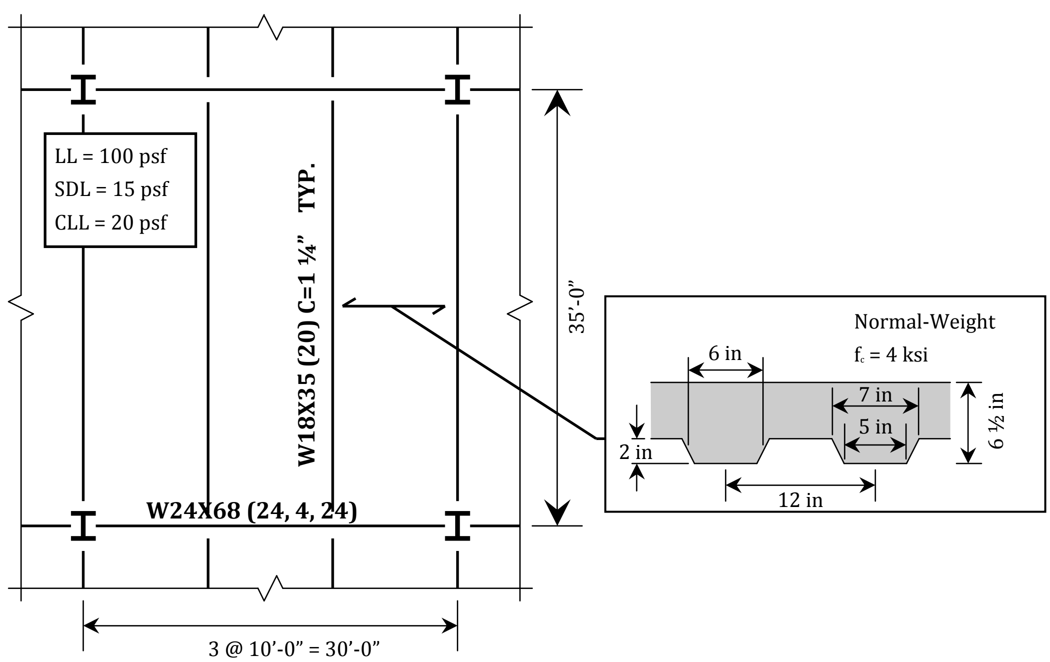


Image from FASTRAK Composite Beam Design

The purpose of this document is to help you quickly build confidence when using FASTRAK. This document shows the long-hand engineering for the LRFD Girder Design tutorial example provided in the installation. This same example is used in the written and video tutorials accompanying FASTRAK Composite Beam (available at http://www.cscworld.com/fastrak/us/composite_resources.html).

This document was produced using the TEDDS calculation software.

Design Details



BASIC DATA

Typical Interior Girder: **W24X68 (24, 4, 24)**

Beam Length (Girder Spacing)	L_{bm} = 35 ft and $S_{gr} = L_{bm}$
Beam Spacing	S_{bm} = 10 ft
Beam Size	**W 18x35**
Girder Length	L_{gr} = 30 ft
Girder Size	**W 24x68**
Steel yield strength	F_y = 50 ksi
Steel Modulus of elasticity	E_s = 29000 ksi
Beam weight	$Weight_{\_BM}$ = **35.0** plf
Girder weight	$Weight_{\_GR}$ = **68.0** plf

Applied Floor Loads

Live Load	F_{LL} =100 psf - Unreduced
Long-term portion	ρ_{LL_lt} = 33.0%
Long-term distributed live load	$F_{LL\_lt} = \rho_{LL\_lt} \times F_{LL}$ = **33.0** psf
Short-term distributed live load	$F_{LL\_st} = (1-\rho_{LL\_lt}) \times F_{LL}$ = **67.0** psf
Superimposed Dead Load	F_{SDL} = 15 psf
Construction Live Load	F_{CLL} = 20 psf

Concrete Slab and Metal Deck

Metal Deck spans parallel to the girder.

Metal Deck Height	h_r = 2 in
Average width of concrete rib	w_r = 6 in
Concrete rib spacing	s_r = 12 in
Width at top of concrete rib	w_{rt} = 7 in
Width at bottom of concrete rib	w_{rb} = 5 in
Metal Deck weight	F_{md} = 2.61 psf
Topping (above metal deck)	t_c = 4.5 in
Concrete compressive strength	f_c = 4000 psi
Wet concrete density	w_{c_wet} = 150 lb/ft^3
Dry concrete density	w_{c_dry} = 145 lb/ft^3
Short-term concrete modulus of elasticity	$E_{c\_st} = w_{c\_dry}^{1.5} \times \sqrt{f_c}$ = **3492** ksi
Long-term to short-term Modulus ratio	ρ_{Ec} = 0.5
Long-term concrete modulus of elasticity	$E_{c\_lt} = E_{c\_st} \times \rho_{Ec}$ = **1746** ksi
Weight of wet concrete slab	$F_{c\_wet} = (t_c+h_r/2) \times w_{c\_wet}$ = **68.7** psf
Weight of dry concrete slab	$F_{c\_dry} = (t_c+h_r/2) \times w_{c\_dry}$ = **66.5** psf

Design Criteria

Bending resistance factor – steel section	ϕ_{b_steel} = 0.90	AISC 360-05 F1.1
Bending resistance factor – composite section	ϕ_{b_comp} = 0.90	AISC 360-05 I3.2a

For this example, it is assumed that the metal deck DOES NOT brace the top flange during the construction stage. Girder is braced at the locations of the supported beams at 10 ft along the girder.

Unbraced length L_b = 10 ft

Calculate Lateral-torsional buckling modification factor at each critical location

Do not Camber the girder

Deflection Limits

Total Construction $\Delta_{tot\_const\_max} = L_{gr}/240 = \mathbf{1.50}$ in

Composite stage

Slab loads $\Delta_{slab\_comp\_max} = L_{gr}/240 = \mathbf{1.50}$ in

Live Loads $\Delta_{LL\_comp\_max} = L_{gr}/360 = \mathbf{1.00}$ in

Total $\Delta_{tot\_comp\_max} = L_{gr}/240 = \mathbf{1.50}$ in

Studs

Stud Diameter $stud_{dia} = 0.75$ in

Stud Tensile strength $F_u = 65$ ksi

Absolute minimum composite action is 50%, Advisory minimum composite is 50%

Beam Line Loads

Beam weight $Weight_{\_BM} = \mathbf{35.0}$ plf

Slab and Deck

Wet Slab $w_{slab\_wet} = (F_{c\_wet} + F_{md}) \times S_{bm} = \mathbf{714}$ plf

Dry Slab $w_{slab\_dry} = (F_{c\_dry} + F_{md}) \times S_{bm} = \mathbf{691}$ plf

Live

Full $w_{LL} = F_{LL} \times S_{bm} = \mathbf{1000}$ plf

Long-term $w_{LL\_lt} = F_{LL\_lt} \times S_{bm} = \mathbf{330}$ plf

Short-term $w_{LL\_st} = F_{LL\_st} \times S_{bm} = \mathbf{670}$ plf

Superimposed Dead Load $w_{SDL} = F_{SDL} \times S_{bm} = \mathbf{150}$ plf

Construction Live Load $w_{CLL} = F_{CLL} \times S_{bm} = \mathbf{200}$ plf

Girder Point Loads

Beam Dead $P_{beam\_dead} = Weight_{\_BM} \times L_{bm} = \mathbf{1.3}$ kips

Slab and Deck

Wet Slab $P_{slab\_wet} = w_{slab\_wet} \times L_{bm} = \mathbf{25.0}$ kips

Dry Slab $P_{slab\_dry} = w_{slab\_dry} \times L_{bm} = \mathbf{24.2}$ kips

Live

Full $P_{LL} = w_{LL} \times L_{bm} = \mathbf{35.0}$ kips

Long-term $P_{LL\_lt} = w_{LL\_lt} \times L_{bm} = \mathbf{11.6}$ kips

Short-term $P_{LL\_st} = w_{LL\_st} \times L_{bm} = \mathbf{23.5}$ kips

Superimposed Dead Load $P_{SDL} = w_{SDL} \times L_{bm} = \mathbf{5.3}$ kips

Construction Live Load $P_{CLL} = w_{CLL} \times L_{bm} = \mathbf{7.0}$ kips

Design Loads (LRFD)

Dead Load strength combination factor $f_{DL\_st} = 1.2$

Live Load strength combination factor $f_{LL\_st} = 1.6$

Construction Stage Point Load (uses wet slab weight) $P_{r\_const} = f_{DL\_st} \times (P_{beam\_dead} + P_{slab\_wet}) + f_{LL\_st} \times (P_{CLL}) = \mathbf{42.8}$ kips

Construction Stage Line Load $w_{r\_const} = f_{DL\_st} \times (Weight_{\_GR}) = \mathbf{81.6}$ plf

Composite Stage Point Load (uses dry slab weight) $P_{r\_comp} = f_{DL\_st} \times (P_{beam\_dead} + P_{slab\_dry} + P_{SDL}) + f_{LL\_st} \times (P_{LL}) = \mathbf{93.0}$ kips

Composite Stage Line Load $w_{r\_comp} = f_{DL\_st} \times (Weight_{\_GR}) = \mathbf{81.6}$ plf

CONSTRUCTION STAGE

Construction Stage Design Checks – Shear (Girder End)

Required Shear Strength $\mathbf{V_{r\_const} = P_{r\_const} + w_{r\_const} \times (L_{gr}/2) = 44.0}$ **kips**

Web slenderness ratio $h\_to\_t_w = 52.0$

Compact web maximum slenderness ratio $h\_to\_t_{w\_max} = 2.24 \times \sqrt{(E_s/F_y)} = 53.9$

$h\_to\_t_w < h\_to\_t_{w\_max}$ therefore AISC 360-05 G2.1(a) and (G2-2) apply and $C_v = 1.0$

Shear resistance factor – steel only $\phi_{v\_steel} = 1.00$

Web area $A_w = 9.84$ in^2

Nominal shear strength $V_n = 0.6 \times F_y \times A_w \times C_v = 295.2$ kips (G2-1)

Available shear strength $\mathbf{V_c = \phi_{v\_steel} \times V_n = 295.2}$ **kips**

$V_c > V_{r\_const}$ therefore construction stage shear strength is OK

Construction Stage Design Checks – Flexure (Girder Centerline)

Required flexural strength $\mathbf{M_{r\_const} = P_{r\_const} \times S_{bm} + w_{r\_const} \times (L_{gr}^2/8) = 436.8}$ **kip_ft**

The W24X68 section is doubly symmetric and has compact web and flanges in flexure (see User Note AISC360-05 F2), therefore section F2 applies.

Unbraced length $L_b = 10.0$ ft

Radius of gyration $r_y = 1.87$ in

Flexural shape factor $c = 1$

Moment of inertia about y-axis $I_y = 70.4$ in^4

Warping constant $C_w = 9430$ in^6

Section Modulus $S_x = 154.0$ in^3

Effective radius of gyration

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}}$$

$r_{ts} = 2.30$ in

Torsional constant $J = 1.87$ in^4

Steel Girder depth $d_s = 23.70$ in

Steel Girder flange thickness $t_f = 0.59$ in

Distance between flange centroids $h_o = d_s - t_f = 23.12$ in

Limiting unbraced length for yielding

$$L_p = 1.76 r_y \sqrt{\frac{E_s}{F_y}}$$

$L_p = 6.61$ ft

Limiting unbraced length for inelastic lateral-torsional buckling

$$L_r = 1.95 r_{ts} \frac{E_s}{.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{.7F_y}{E_s} \frac{S_x h_o}{Jc} \right)}}$$

$L_r = 18.86$ ft

The unbraced length, L_b, is greater than L_p and less than L_r, therefore the limit states of Yielding and Lateral-Torsional Buckling (LTB) apply (AISC 360-05 F2.2) and the nominal flexural strength is determined by (F2-2)

LTB modification factor

$M_{max} = M_{r\_const} =$ **436.8** kip_ft

At quarter point $l_A = 12.5$ ft $M_A = P_{r\_const} \times S_{bm} + w_{r\_const} \times l_A / 2 \times (L_{gr} - l_A) =$ **436.5** kip_ft

At centerline $M_B = M_{r\_const} =$ **436.8** kip_ft

At three-quarter point $l_C = 17.5$ ft $M_C = P_{r\_const} \times S_{bm} + w_{r\_const} \times l_C / 2 \times (L_{gr} - l_C) =$ **436.5** kip_ft

Cross-section monosymmetry parameter $R_m = 1.0$ doubly symmetric member

Lateral-torsional buckling modification factor

$$C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} R_m$$

$C_b =$ **1.00**

Plastic Section Modulus $Z_x =$ **177.0** in^3

Plastic Flexural Strength $M_p = F_y \times Z_x =$ **737.5** kip_ft (F2-1)

Nominal Flexural Strength per (F2-2)

$$M_{n\_F2\_2} = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]$$

$M_{n\_F2\_2} =$ **657.8** kip_ft

Nominal Flexural Strength $M_{n\_const} = \text{Min}(M_p, M_{n\_F2\_2}) =$ **657.8** kip_ft

Available Flexural Strength **$M_{c\_const} = \phi_{b\_steel} \times M_{n\_const} =$ 592.0 kip_ft**

$M_{c\_const} > M_{r\_const}$ therefore construction stage flexural strength is OK

The unbraced lengths from the girder ends to the location of the supported beams (10 ft from girder end) need also be checked. However, the unbraced length is 10 ft and C_b will be greater than one due to the non-uniform moment in these regions. Therefore the available strength will be greater than or equal to that calculated for the center region. The required flexural strength will be essentially the same as at the centerline. Therefore the end regions are OK for the construction stage flexure as well. The details of the calculation are not shown.

Construction Stage Design Checks – Deflection (Beam Centerline)

Moment of Inertia of bare steel beam $I_x =$ **1830.0** in^4

Dead Load deflection - due to girder self weight, supported beam weight and slab wet (includes metal deck weight)

$\Delta_{DL\_const} = 5 \times (\text{Weight}_{GR}) \times L_{gr}^4/(384 \times E_s \times I_x) + (P_{slab\_wet} + P_{beam\_dead}) \times L_{gr}^3/(28 \times E_s \times I_x) =$ **0.85** in

Camber = 0 in

Construction Live load deflection $\Delta_{LL\_const} = P_{CLL} \times L_{gr}^3/(28 \times E_s \times I_x) =$ **0.22** in

Total construction stage deflection **$\Delta_{tot\_const} = (\Delta_{DL\_const} - \text{Camber}) + \Delta_{LL\_const} =$ 1.07 in**

Construction Stage Deflection Limit $\Delta_{tot\_const\_max} =$ **1.50** in

$\Delta_{tot\_const} > \Delta_{tot\_const\_max}$ therefore construction stage deflection OK

COMPOSITE STAGE

Composite Stage Design Checks – Shear (Girder End)

Required Shear Strength **$V_{r\_comp} = P_{r\_comp} + w_{r\_comp} \times (L_{gr}/2) =$ 94.2 kips**

Shear strength for composite section is based on the bare steel beam only (AISC 360-05 I3.1b), therefore Chapter G applies and the nominal and available shear strengths are the same as those for the construction stage.

Nominal shear strength $V_n =$ **295.2** kips (G2-1)

Available shear strength **$V_c = \phi_{v\_steel} \times V_n =$ 295.2 kips**

$V_c > V_{r\_comp}$ therefore shear strength is OK

Composite Stage Design– Flexure (General)

Web slenderness ratio $h\_to\_t_w$ = **52.0**

Web maximum slenderness ratio $h\_to\_t_{w\_maxcomp} = 3.76 \times \sqrt{(E_s/F_y)}$ = **90.6**

$h\_to\_t_w < h\_to\_t_{w\_maxcomp}$ therefore AISC 360-05 I3.2a(a) applies and the nominal flexural strength of the composite section can be determined from the plastic stress distribution on the composite section

Effective concrete width $b_{eff} = \mathrm{Min}(2 \times L_{gr}/8, 2 \times S_{gr}/2)$ = **90.0** in

Effective area of concrete $A_c = (b_{eff} \times t_c) + [h_r \times (w_{rt} + w_{rb})/2] \times (b_{eff}/s_r)$ = **495.0** in^2

Concrete below top of deck is included in composite properties for parallel metal deck [AISC 360-05 I3.2c(2)]. The equation for A_c includes the area of one rib multiplied by the approximate number of ribs, (b_{eff}/s_r).

Area of steel beam A_s = **20.1** in^2

Stud strength – any number of studs per group

Group Factor: Any number of studs welded in a row through the steel deck with the deck oriented parallel to the steel shape and the ratio of average rib width to rib depth, w_r/h_r = **3.00**, is greater than 1.5 (AISC 360-05 I3.2d(3))

$R_g = 1.0$

Position Factor: Studs welded in a composite slab with the deck oriented parallel to the beam (AISC 360-05 I3.2d(3)

$R_p = 0.75$

Nominal Stud Strength

Cross-sectional area of shear connector $A_{sc} = \pi \times (stud_{dia}/2)^2$ = **0.44** in^2

Nominal strength based on concrete $Q_{n\_conc} = 0.5 \times A_{sc} \times \sqrt{(f_c \times E_{c\_st})}$ = **26.1** kips AISC 360-05 (I3-3)

Nominal strength based on geometry $Q_{n\_geom} = R_g \times R_p \times A_{sc} \times F_u$ = **21.5** kips AISC 360-05 (I3-3)

Nominal strength of one stud $Q_n = \mathrm{Min}(Q_{n\_conc}, Q_{n\_geom})$ = **21.5** kips

Minimum longitudinal stud spacing $S_{st\_min} = 6 \times stud_{dia}$ = **4.50** in

Maximum longitudinal stud spacing $S_{st\_max} = \mathrm{Min}(8 \times [t_c + h_r], 36\text{ in})$ = **36.00** in

Minimum transverse stud spacing $S_{st\_trans\_min} = 4 \times stud_{dia}$ = **3.00** in

Girder flange width b_f = **8.97** in

Minimum flange width assuming 1.5 in edge distance $b_{f\_min} = S_{st\_trans\_min} + 2 \times 1.5\text{ in}$ = **6.0** in

$b_f > b_{f\_min}$ therefore two studs per row OK

Horizontal shear at beam-slab interface

Shear - Concrete Crushing $V_{p\_concrete\_crushing} = 0.85 \times f_c \times A_c$ = **1683.0** kips

Shear – Steel Yielding $V_{p\_steel\_yield} = F_y \times A_s$ = **1005.0** kips

Shear at full interaction $V_{p\_Full} = \mathrm{Min}(V_{p\_concrete\_crushing}, V_{p\_steel\_yield})$ = **1005.0** kips

Composite Section Properties

The steel section is idealized as a series of three rectangles. The total area of the steel section is maintained by incorporating the area of the fillet radius into the flanges. This is accomplished by increasing the width of the top and bottom flange.

Steel girder depth d_s = **23.70** in

Steel girder web thickness t_w = **0.41** in

Steel girder flange thickness t_f = **0.59** in

Area of steel girder web $A_{web} = (d_s - 2 \times t_f) \times t_w$ = **9.35** in^2

Steel girder flange width b_f = **8.97** in

Effective area of each flange for use in composite section calculations

$A_{f\_eff} = (A_s - A_{web})/2 =$ **5.38** in^2

Effective width of flanges for use in composite section calculations

$b_{f\_eff} = A_{f\_eff}/t_f =$ **9.19** in

Tensile Strength of steel $P_y = V_{p\_steel\_yield} =$ **1005.0** kips

Max compression force in steel flange $C_{steel\_flange\_max} = F_y \times t_f \times b_{f\_eff} =$ **268.8** kips

Composite Stage Design Checks – Flexure (@ Point Loads)

Distance from Girder End to Critical Location $d_{crit\_PL} = 10$ ft

Required flexural strength $\mathbf{M_{r\_comp\_PL} = P_{r\_comp} \times d_{crit\_PL} + w_{r\_comp} \times d_{crit\_PL}/2 \times (L_{gr} - d_{crit\_PL}) =}$ **937.8 kip_ft**

Number of Studs from beam end to 10 ft from girder end, two studs per row $\mathbf{N_{studs\_PL} = 24}$

Stud spacing $S_{st\_PL} = d_{crit\_PL}/(N_{studs\_PL}/2) =$ **10.00** in

$S_{st\_max} > S_{st\_PL} > S_{st\_min}$ therefore stud spacing OK

Horizontal shear at beam-slab interface

Shear in Studs $V_{p\_studs\_PL} = N_{studs\_PL} \times Q_n =$ **516.9** kips

Horizontal shear $\mathbf{V_{p\_PL} = Min(V_{p\_studs\_PL}, V_{p\_concrete\_crushing}, V_{p\_steel\_yield}) =}$ **516.9 kips**

Percent composite action $Comp_{percent\_PL} = V_{p\_PL}/V_{p\_Full} =$ **51.4** %

$Comp_{percent\_PL}$ is greater than the absolute minimum (50%) – OK

$Comp_{percent\_PL}$ is less than the advisory minimum (50%) – OK

Composite Section Properties (@ Point Loads)

Compression force in concrete $C_{conc\_PL} = V_{p\_PL} =$ **516.9** kips

Effective depth of concrete in compression $a_{eff\_PL} = C_{conc\_PL}/(0.85 \times f_c \times b_{eff}) =$ **1.69** in

The effective depth is less than the concrete topping, therefore there is no contribution from the ribs and the equation above is valid. In addition, the equations for d_{1_PL} and d_{1_CL} can neglect the rib contribution.

Compression in Steel beam $C_{steel\_PL} = (P_y - C_{conc\_PL})/2 =$ **244.1** kips

Max compression force in steel flange $C_{steel\_flange\_max} =$ **268.8** kips

$C_{steel\_PL} < C_{steel\_flange\_max}$ therefore plastic neutral axis is in the beam flange and $C_{steel\_flange\_PL} = C_{steel\_PL}$

Compression force in the beam web $C_{steel\_web\_PL} = 0$ kips

Distance (down) of location of plastic neutral axis from top of steel beam

$PNA_{\_PL} = C_{steel\_flange\_PL}/(b_{f\_eff} \times F_y) =$ **0.53** in

Nominal Moment Strength is determined using Figure C-I3.1 (shown below) and Equation(C-I3-5) from the Commentary to AISC LRFD Specification for Structural Steel Buildings 1999. See Figure 1.

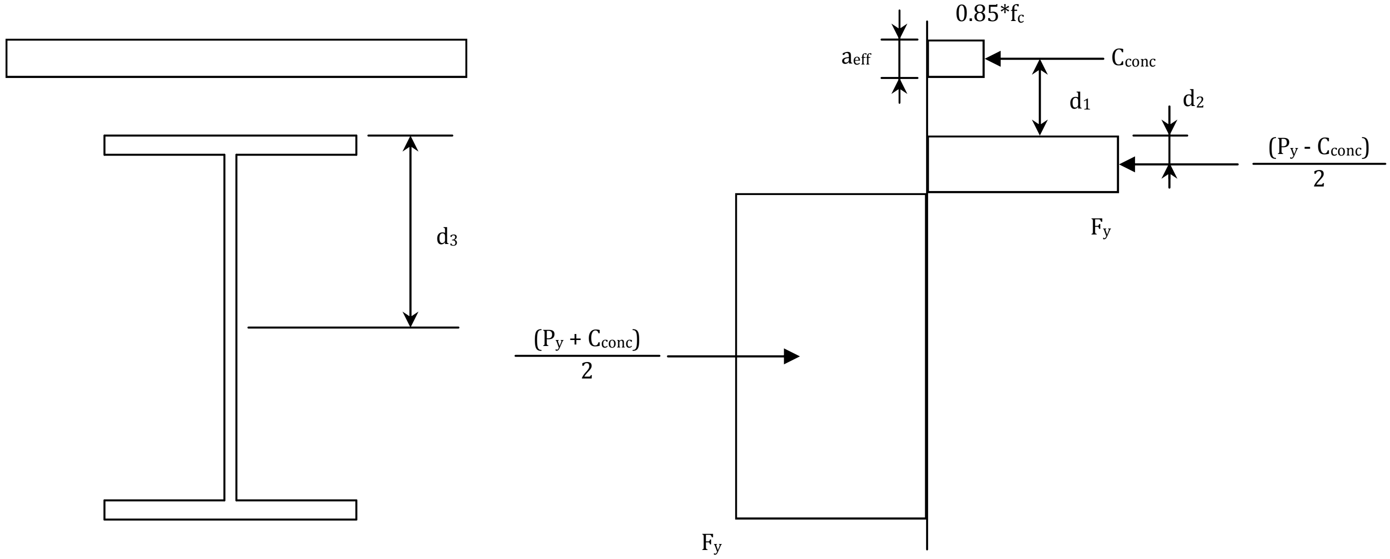


Figure 1: Commentary to the AISC LRFD Specification for Structural Steel Buildings 1999—Fig. C-I3.1: Plastic Stress distribution for positive moment in composite beams.

Distance from the centroid of the compression force in the concrete to the top of the steel section

$d_{1\_PL} = (h_r + t_c) - a_{eff\_PL}/2 =$ **5.66** in

Distance from the centroid of the compression force in the steel section to the top of the steel section

$d_{2\_PL} = (C_{steel\_flange\_PL} \times PNA_{\_PL}/2)/ C_{steel\_PL} =$ **0.27** in

Distance from the centroid of the steel section (and P_y) to the top of the steel section

$d_{3\_PL} = d_s/2 =$ **11.85** in

Nominal Composite Flexural Strength $M_{n\_comp\_PL} = C_{conc\_PL} \times (d_{1\_PL} + d_{2\_PL}) + P_y \times (d_{3\_PL} - d_{2\_PL}) =$ **1225.2** kip_ft

Available Composite Flexural Strength $\mathbf{M_{c\_comp\_PL} = \phi_{b\_comp} \times M_{n\_comp\_PL} =}$ **1102.7 kip_ft**

$M_{c\_comp\_PL}$ > $M_{r\_comp\_PL}$ therefore shear strength is OK

The same design checks apply at 20 ft from the left end of the girder (the location of the other supported beams) and are not repeated.

<u>Composite Stage Design Checks – Flexure (Girder Centerline)</u>

Distance from Girder End to Critical Location $d_{crit\_CL} = L_{gr}/2 =$ **15.00** ft

Required flexural strength $\mathbf{M_{r\_comp\_CL} = P_{r\_comp} \times d_{crit\_PL} + w_{r\_comp} \times (L_{gr}^2/8) =}$ **938.8 kip_ft**

Number of Studs from girder end to centerline, $\mathbf{N_{studs\_CL} = 26}$

24 (two studs per row) from girder end to 10 ft from girder end and 2 from 10 ft from girder end to girder centerline

Stud spacing, center region $S_{st\_CL} = (d_{crit\_CL} - d_{crit\_PL}) / (N_{studs\_CL} - N_{studs\_PL}) =$ **30.00** in

S_{st_max} > S_{st_CL} > S_{st_min} therefore stud spacing OK

Horizontal shear at beam-slab interface

Shear in Studs $V_{p\_studs\_CL} = N_{studs\_CL} \times Q_n =$ **560.0** kips

Horizontal shear $\mathbf{V_{p\_CL} = Min(V_{p\_studs\_CL}, V_{p\_concrete\_crushing}, V_{p\_steel\_yield}) =}$ **560.0 kips**

Percent composite action $Comp_{percent\_CL} = V_{p\_CL}/V_{p\_Full} =$ **55.7** %

$Comp_{percent\_CL}$ is greater than the absolute minimum (50%) – OK

$Comp_{percent\_CL}$ is less than the advisory minimum (50%) – OK

Composite Section Properties (Girder Centerline)

Compression force in concrete $C_{conc\_CL} = V_{p\_CL} =$ **560.0** kips

Effective depth of concrete in compression $a_{eff\_CL} = C_{conc\_CL}/(0.85 \times f_c \times b_{eff}) =$ **1.83** in

Compression in Steel beam $C_{steel\_CL} = (P_y - C_{conc\_CL})/2 =$ **222.5** kips

Max compression force in steel flange $C_{steel\_flange\_max} =$ **268.8** kips

$C_{steel\_CL} < C_{steel\_flange\_max}$ therefore plastic neutral axis is in the beam flange and $C_{steel\_flange\_CL} = C_{steel\_CL}$

Compression force in the beam web $C_{steel\_web\_CL} = 0$ kips

Distance (down) of location of plastic neutral axis from top of steel beam

$PNA_{\_CL} = C_{steel\_flange\_CL}/(b_{f\_eff} \times F_y) =$ **0.48** in

Nominal Moment Strength is determined using Figure C-I3.1 (shown below) and Equation(C-I3-5) from the Commentary to AISC LRFD Specification for Structural Steel Buildings 1999. See Figure 1

Distance from the centroid of the compression force in the concrete to the top of the steel section

$d_{1\_CL} = (h_r + t_c) - a_{eff\_CL}/2 =$ **5.59** in

Distance from the centroid of the compression force in the steel section to the top of the steel section

$d_{2\_CL} = (C_{steel\_flange\_CL} \times PNA_{\_CL}/2)/\ C_{steel\_CL} =$ **0.24** in

Distance from the centroid of the steel section (and P_y) to the top of the steel section

$d_{3\_CL} = d_s/2 =$ **11.85** in

Nominal Composite Flexural Strength $M_{n\_comp\_CL} = C_{conc\_CL} \times (d_{1\_CL} + d_{2\_CL}) + P_y \times (d_{3\_CL} - d_{2\_CL}) =$ **1244.1** kip_ft

Available Composite Flexural Strength $\mathbf{M_{c\_comp\_CL} = \phi_{b\_comp} \times M_{n\_comp\_CL} =}$ **1119.7 kip_ft**

$M_{c\_comp\_CL} > M_{r\_comp\_CL}$ therefore shear strength is OK

<u>Composite Stage Design Checks – Elastic Section Properties (Girder Centerline)</u>

Steel Girder Moment of Inertia $I_x =$ **1830.0** in^4

Steel Girder Area $A_s =$ **20.10** in^2

Area of Concrete $A_c =$ **495.0** in^2

The concrete is treated as a solid rectangle above the deck and as a series of trapezoids within the deck ribs. The properties of one trapezoid are then multiplied by the effective number of trapezoids within the effective width, b_{eff}/s_r. See Figure 2.

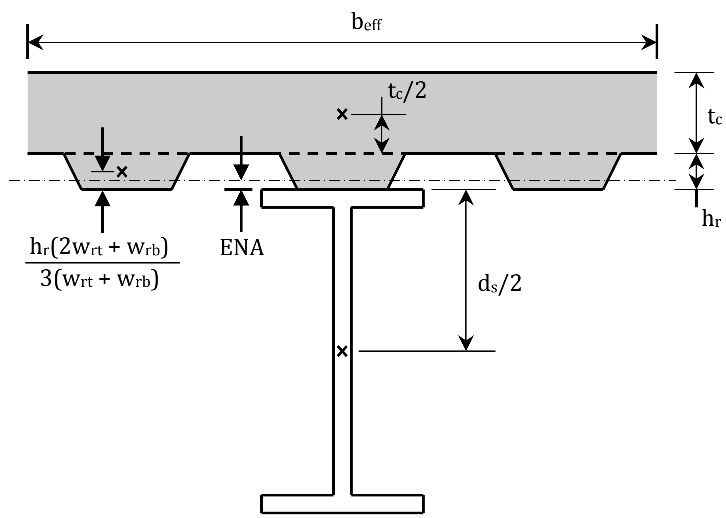


Figure 2: Elastic Composite Section

Depth from bottom of concrete to centroid of concrete

$$d_c = \frac{(b_{eff} \times t_c) \times (h_r + t_c/2) + \left(\frac{h_r(w_{rt}+w_{rb})}{2} \times \frac{h_r(2w_{rt}+w_{rb})}{3(w_{rt}+w_{rb})}\right) \times \frac{b_{eff}}{s_r}}{A_c}$$

d_c = **3.67** in

Moment of inertia of the concrete about its centroid

$$I_c = \frac{b_{eff}t_c^3}{12} + (b_{eff}t_c)[h_r + t_c/2 - d_c]^2 + \left\{\frac{h_r^3(w_{rt}^2 + 4w_{rt}w_{rb} + w_{rb}^2)}{36(w_{rt}+w_{rb})} + \left(\frac{h_r(w_{rt}+w_{rb})}{2}\right)\left[d_c - \frac{h_r(2w_{rt}+w_{rb})}{3(w_{rt}+w_{rb})}\right]^2\right\} \times \left(\frac{b_{eff}}{s_r}\right)$$

I_c = **1465** in^4

Short-term modular ratio $n_{st} = E_s/E_{c\_st}$ = **8.3**

Short-term Elastic neutral axis (up from top of steel beam)

$$ENA_{st} = \frac{A_c/n_{st} \times d_c - A_s \times d_s/2}{A_c/n_{st} + A_s}$$

ENA_{st} = **-0.24** in

Short-term transform moment of inertia taken about the elastic neutral axis

$$I_{tr\_st} = I_x + A_s\left({}^{d_s}/_2 + ENA_{st}\right)^2 + {}^{I_c}/_{n_{st}} + \left({}^{A_c}/_{n_{st}}\right)[d_c - ENA_{st}]^2$$

I_{tr_st} = **5627** in^4

Short-term transform moment of inertia with correction for deviation from elastic theory AISC 360-05 Commentary C-I3.1

$I_{tr\_eff\_st} = 0.75 \times I_{tr\_st}$ = **4220** in^4

Short-term effective moment of inertia due to partial composite action AISC 360-05 Commentary (C-I3-3), V_{p_CL} at centerline

$$I_{eff\_st} = I_x + (I_{tr\_eff\_st} - I_x)\sqrt{V_{p\_CL}/V_{p\_Full}}$$

I_{eff_st} = **3614** in^4

Long-term modular ratio $n_{lt} = E_s/E_{c\_lt}$ = **16.6**

Long-term Elastic neutral axis (up from top of steel beam)

$$ENA_{lt} = \frac{A_c/n_{lt} \times d_c - A_s \times d_s/2}{A_c/n_{lt} + A_s}$$

ENA_{lt} = **-2.58** in

Long-term transform moment of inertia taken about elastic neutral axis

$$I_{tr\_lt} = I_x + A_s\left({}^{d_s}/_2 + ENA_{lt}\right)^2 + {}^{I_c}/_{n_{lt}} + \left({}^{A_c}/_{n_{lt}}\right)[d_c - ENA_{lt}]^2$$

I_{tr_lt} = **4809** in^4

Long -term transform moment of inertia with correction for deviation from elastic theory AISC 360-05 Commentary C-I3.1

$I_{tr\_eff\_lt} = 0.75 \times I_{tr\_lt}$ = **3607** in^4

Long -term effective moment of inertia due to partial composite action AISC 360-05 Commentary (C-I3-3), V_{p_CL} at centerline

$$I_{eff\_lt} = I_x + (I_{tr\_eff\_lt} - I_x)\sqrt{V_{p\_CL}/V_{p\_Full}}$$

I_{eff_lt} = **3156** in^4

Composite Stage Design Checks – Deflections

Camber = **0.00** in

Slab loads (Beam weight and dry slab weight, including metal deck and camber) on steel beam

Girder weight (self weight) $\Delta_{Girder} = 5 \times (\text{Weight}_{GR}) \times L_{gr}^4/(384 \times E_s \times I_x)$ = **0.02** in

Beam Dead $\Delta_{beam\_dead} = (P_{beam\_dead}) \times L_{gr}^3/(28 \times E_s \times I_x)$ = **0.04** in

Dry slab weight only $\Delta_{slab\_only} = (P_{slab\_dry}) \times L_{gr}^3/(28 \times E_s \times I_x)$ = **0.76** in

Total Slab $\Delta_{slab\_total} = \Delta_{Girder} + \Delta_{beam\_dead} + \Delta_{slab\_only}$ = **0.82** in

Slab Adjusted for Camber $\Delta_{slab} = \Delta_{slab\_total} - \text{Camber}$ = **0.82 in**

Slab Deflection Limit $\Delta_{slab\_comp\_max}$ = **1.50** in

$\Delta_{slab\_comp\_max} > \Delta_{slab}$ therefore slab load deflection is OK

Live Loads (take into account long- and short-term concrete moduli and loads) on composite section

Short-term live load deflection $\Delta_{LL\_st} = (P_{LL\_st}) \times L_{gr}^3/(28 \times E_s \times I_{eff\_st})$ = **0.37** in

Long-term live load deflection $\Delta_{LL\_lt} = (P_{LL\_lt}) \times L_{gr}^3/(28 \times E_s \times I_{eff\_lt})$ = **0.21** in

Total live load deflection $\Delta_{LL} = \Delta_{LL\_st} + \Delta_{LL\_lt}$ = **0.58 in**

Live Load Deflection Limit $\Delta_{LL\_comp\_max}$ = **1.00** in

$\Delta_{LL\_comp\_max} > \Delta_{LL}$ therefore live load deflection is OK

Dead Load (all load considered long-term) on composite section

Superimposed Dead $\Delta_{SDL} = (P_{SDL}) \times L_{gr}^3/(28 \times E_s \times I_{eff\_lt})$ = **0.10 in**

Total Deflection

Total Deflection (incl. Camber) $\Delta_{tot\_comp} = \Delta_{slab} + \Delta_{LL} + \Delta_{SDL}$ = **1.50 in**

Total Deflection Limit $\Delta_{tot\_comp\_max}$ = **1.50** in

$\Delta_{tot\_comp\_max} \geq \Delta_{tot\_comp}$ therefore total deflection is OK

In these calculations for deflection, the weight of the supported beams is applied to the bare steel girder. In the sample file provided with FASTRAK Composite Beam Design a slightly different approach is taken. The beam weight is added as a standard dead load case and the deflection due to this load is applied to the composite section as a long-term load. This results in a small difference in the deflections (as shown below) from the calculations included above. The loads were added as a separate load case to simplify the load input and to clearly indicate that the supported steel weight was included. To include the beam weight as it is in these calculations, it can be added to the 'slab wet' and 'slab dry' load cases instead of its own additional dead load case. The following deflections correspond to the manner in which the loads were added in FASTRAK Composite Beam Design (FCBD.)

Girder weight (self weight) $\Delta_{Girder} = 5 \times (Weight_{GR}) \times L_{gr}^4/(384 \times E_s \times I_x)$ = **0.02** *in*

Total Slab (not including steel weight) $\Delta_{slab\_total\_FCBD} = \Delta_{slab\_only}$ = **0.76** *in*

Slab Adjusted for Camber $\Delta_{slab\_FCBD} = \Delta_{slab\_total\_FCBD} - Camber$ = **0.76** *in*

Beam Dead – applied to composite section $\Delta_{beam\_dead\_FCBD} = (P_{beam\_dead}) \times L_{gr}^3/(28 \times E_s \times I_{eff\_lt})$ = **0.02** *in*

Superimposed Dead (Dead) Δ_{SDL} = **0.10** *in*

Total Dead load on composite section $\Delta_{dead\_comp\_FCBD} = \Delta_{beam\_dead\_FCBD} + \Delta_{SDL}$ = **0.12** *in*

Total live load deflection Δ_{LL} = **0.58** *in*

Total deflection $\Delta_{tot\_comp\_FCBD} = \Delta_{slab\_FCBD} + \Delta_{LL} + \Delta_{Girder} + \Delta_{dead\_comp\_FCBD}$ = ***1.49 in***

Note: If you are working with FASTRAK Building Designer to establish and compare these beam examples, all the steel weight is automatically included in the 'self weight' load case and is applied to the bare steel girder as indicated in these calculations. All the geometry, flange bracing, floor construction, and loading data is generated by FASTRAK Building Designer within the full building model and can be automatically exported to the composite beam design component for more detailed analysis.

SUMMARY – W24X68 (24, 4, 24)

Construction Stage

Design Condition	***Critical Value***	***Capacity Limit***	***Ratio***
Vertical Shear (End)	V_{r_const} = **44** kips	V_c = **295** kips	V_{r_const} / V_c = **0.149**
Flexure (Centerline)	M_{r_const} = **437** kip_ft	M_{c_const} = **592** kip_ft	M_{r_const} / M_{c_const} = **0.738**
Deflection (Centerline)	Δ_{tot_const} = **1.07** in	$\Delta_{tot\_const\_max}$ = **1.50** in	Δ_{tot_const} / $\Delta_{tot\_const\_max}$ = **0.713**

Composite Stage

Design Condition	***Critical Value***	***Capacity Limit***	***Ratio***
Vertical Shear (End)	V_{r_comp} = **94** kips	V_c = **295** kips	V_{r_comp} / V_c = **0.319**
Flexure (Point Loads)	$M_{r\_comp\_PL}$ = **938** kip_ft	$M_{c\_comp\_PL}$ = **1103** kip_ft	$M_{r\_comp\_PL}$ / $M_{c\_comp\_PL}$ = **0.850**
Flexure (Centerline)	$M_{r\_comp\_CL}$ = **939** kip_ft	$M_{c\_comp\_CL}$ = **1120** kip_ft	$M_{r\_comp\_CL}$ / $M_{c\_comp\_CL}$ = **0.838**
Deflections	Camber = **0.00** in		
Slab (incl. Camber)	Δ_{slab} = **0.82** in	$\Delta_{slab\_comp\_max}$ = **1.50** in	Δ_{slab} / $\Delta_{slab\_comp\_max}$ = **0.549**
Live	Δ_{LL} = **0.58** in	$\Delta_{LL\_comp\_max}$ = **1.00** in	Δ_{LL} / $\Delta_{LL\_comp\_max}$ = **0.583**
Superimposed Dead	Δ_{SDL} = **0.10** in	NA	
Total	Δ_{tot_comp} = **1.50** in	$\Delta_{tot\_comp\_max}$ = **1.50** in	Δ_{tot_comp} / $\Delta_{tot\_comp\_max}$ = **1.000**

DESIGN METHOD:

There is a direct relationship between the safety factors (Ω) used in ASD and the resistance factors (ϕ) used for LRFD. Namely, $\Omega=1.5/\phi$. When the required strength using LRFD load combinations is about 1.5 times the strength required using ASD load combinations, the design of the two methods will likely be the same. This corresponds to a live load to dead load ratio of 3 for load combinations involving only live and dead loads. When the ratio is less than 3 the ASD method may require larger steel sections or more studs. When the ratio is greater than 3 the LRFD method may require larger steel sections or more studs.

In this example, the composite live to dead load ratio is: $(P_{LL})/(P_{SDL} + P_{slab\_dry} + P_{beam\_dead} + \text{Weight}_{GR} \times L_{gr})$ = **1.07**

This means there is the potential that the ASD method will require a heavier steel section or more studs. However, the overall design for this example girder using the LRFD method of design is the same as when the ASD method is used. This is due to the fact that the number of studs is based on achieving the minimum required composite action of 50% and the fact that the deflection controls the design. The details of the ASD design are presented in the design example entitled “ASD Girder” – available on the online support website: http://www.cscworld.com/fastrak/us/composite_resources.html.