FASTRAK Composite Beam Design

LRFD Beam 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

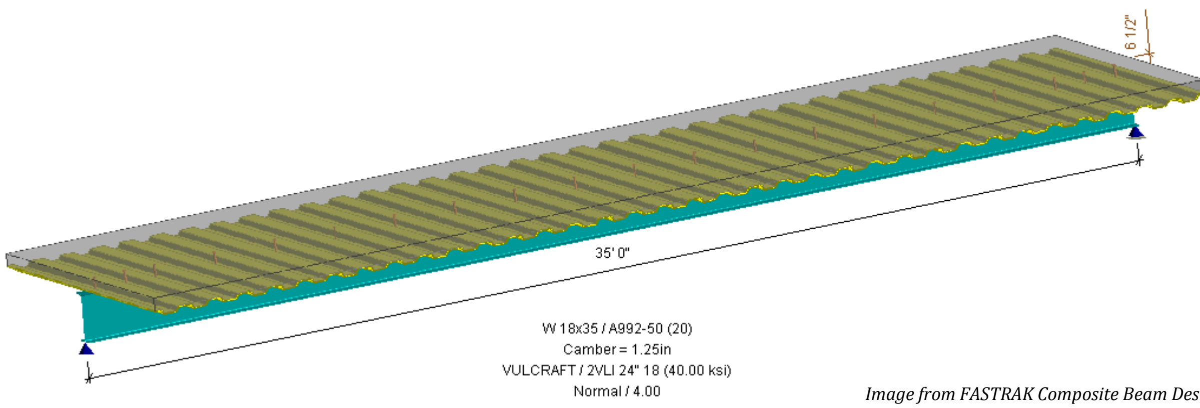
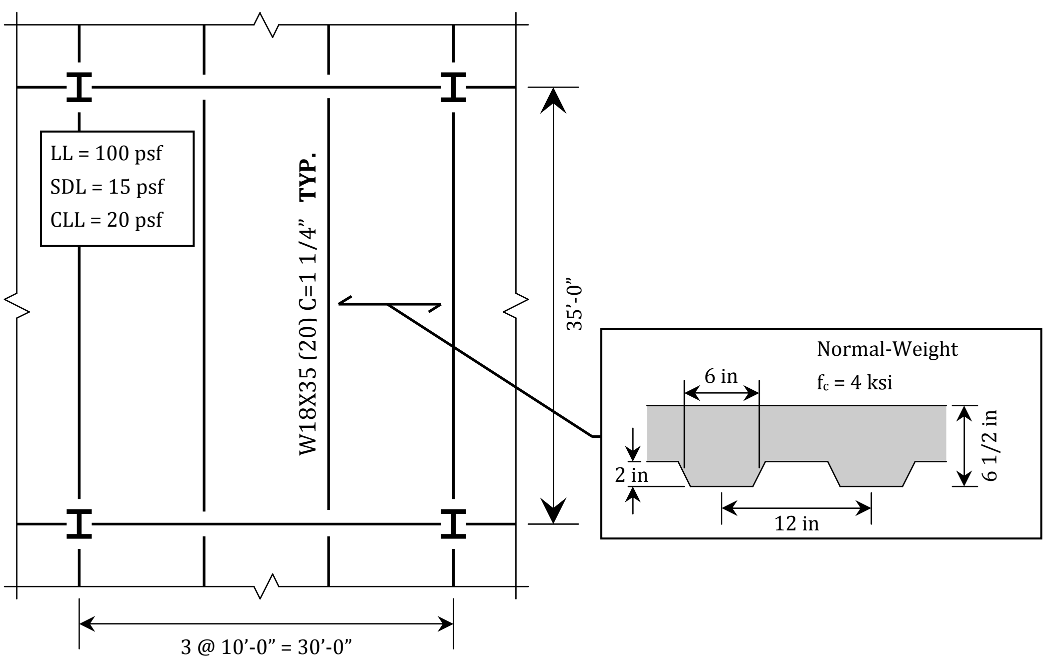


Image from FASTRAK Composite Beam Design

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 Beam 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 Beam: **W18X35 (20) with 1.25 in Camber**

Beam Length	L_{bm} = 35 ft
Beam Spacing	S_{bm} = 10 ft
Beam Size	**W 18x35**
Steel yield strength	F_y = 50 ksi
Steel Modulus of elasticity	E_s = 29000 ksi
Beam weight	$Weight_{\_BM}$ = **35.0** plf

Applied Floor Loads

Live Load	F_{LL} =100 psf - Unreduced
Long-term portion	ρ_{LL_lt} = 33%
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 perpendicular to the beam.

Metal Deck Height	h_r = 2 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 braces top flange continuously during construction stage.

Unbraced length	L_b = 0 ft
Lateral-torsional buckling modification factor	C_b = 1.0

Camber 75% of dead load, apply no less than ¾ in of camber at ¼ in increments

Deflection Limits

Total Construction	$\Delta_{tot\_const\_max} = L_{bm}/240$ = **1.75** in
Composite stage	
Slab loads	$\Delta_{slab\_comp\_max} = L_{bm}/240$ = **1.75** in
Live Loads	$\Delta_{LL\_comp\_max} = L_{bm}/360$ = **1.17** in
Total	$\Delta_{tot\_comp\_max} = L_{bm}/240$ = **1.75** in

Studs

Distance from stud to deck — $e_{mid\text{-}ht} < 2$ in

Stud Diameter — $stud_{dia} = 0.75$ in

Stud Tensile strength — $F_u = 65$ ksi

Absolute minimum composite action is 25%, Advisory minimum composite is 50%

Beam Line Loads

Beam Self 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 — $w_{LL\_lt} = F_{LL\_lt} \times S_{bm} = \mathbf{330.0}$ 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

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 Line Load (uses wet slab weight) — $w_{r\_const} = f_{DL\_st} \times (Weight_{\_BM} + w_{slab\_wet}) + f_{LL\_st} \times (w_{CLL}) = \mathbf{1218}$ plf

Composite Stage Line Load (uses dry slab weight) — $w_{r\_comp} = f_{DL\_st} \times (Weight_{\_BM} + w_{slab\_dry} + w_{SDL}) + f_{LL\_st} \times (w_{LL}) = \mathbf{2651}$ plf

CONSTRUCTION STAGE

Construction Stage Design Checks – Shear (Beam End)

Required Shear Strength — $\mathbf{V_{r\_const} = w_{r\_const} \times (L_{bm}/2) = 21.3}$ **kips**

Web slenderness ratio — $h\_to\_t_w = \mathbf{53.5}$

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

$h\_to\_t_w < h\_to\_t_{w\_max}$ 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 — $A_w = \mathbf{5.31}$ in^2

Nominal shear strength — $V_n = 0.6 \times F_y \times A_w \times C_v = \mathbf{159.3}$ kips (G2-1)

Available shear strength — $\mathbf{V_c = \phi_{v\_steel} \times V_n = 159.3}$ **kips**

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

Construction Stage Design Checks – Flexure (Beam Centerline)

Required flexural strength — $\mathbf{M_{r\_const} = w_{r\_const} \times (L_{bm}^2/8) = 186.6}$ **kip_ft**

The W18X35 section is doubly symmetric and has compact web and flanges in flexure (see User Note AISC360-05 F2), therefore section F2 applies.

The unbraced length, L_b, is equal to zero, therefore only the limit state of Yielding applies (AISC 360-05 F2.2) and the nominal flexural strength is determined by (F2-1)

Plastic Section Modulus — $Z_x = \mathbf{66.5}$ in^3

Nominal Flexural Strength — $M_{n\_const} = F_y \times Z_x = \mathbf{277.1}$ kip_ft (F2-1)

Available Flexural Strength $M_{c\_const} = \phi_{b\_steel} \times M_{n\_const}$ = **249.4 kip_ft**

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

Construction Stage Design Checks – Deflection (Beam Centerline)

Moment of Inertia of bare steel beam I_x = 510.0 in^4

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

Dead load Deflection $\Delta_{DL\_const} = 5 \times (w_{slab\_wet} + Weight_{\_BM}) \times L_{bm}^4/(384 \times E_s \times I_x)$ = 1.71 in

Camber $0.75 \times \Delta_{DL\_const}$ = 1.28 in - therefore Camber = 1.25 in

Construction Live load deflection $\Delta_{LL\_const} = 5 \times (w_{CLL}) \times L_{bm}^4/(384 \times E_s \times I_x)$ = 0.46 in

Total construction stage deflection $\Delta_{tot\_const} = (\Delta_{DL\_const} - Camber) + \Delta_{LL\_const}$ = **0.92 in**

Construction Stage Deflection Limit $\Delta_{tot\_const\_max}$ = 1.75 in

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

COMPOSITE STAGE

Composite Stage Design Checks – Shear (Beam End)

Required Shear Strength $V_{r\_comp} = w_{r\_comp} \times (L_{bm}/2)$ = **46.4 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 = 159.3 kips (G2-1)

Available shear strength $V_c = \phi_{v\_steel} \times V_n$ = **159.3 kips**

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

Composite Stage Design Checks – Flexure (Beam Centerline)

Required flexural strength $M_{r\_comp} = w_{r\_comp} \times (L_{bm}^2/8)$ = **405.9 kip_ft**

Method to Determine Nominal Flexural Strength

Web slenderness ratio $h\_to\_t_w$ = 53.5

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 $b_{eff} = Min(2 \times L_{bm}/8, 2 \times S_{bm}/2)$ = 105.0 in

Effective area of concrete $A_c = b_{eff} \times t_c$ = 472.5 in^2

Concrete below top of deck is not included in composite properties for perpendicular metal deck [AISC 360-05 I3.2c(2)]

Area of steel beam A_s = 10.3 in^2

Shear Interaction (Composite Action)

Stud strength – one stud per rib

Group Factor: One stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape (AISC 360-05 I3.2d(3)) $R_g = 1.0$

Position Factor: Studs welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 2$ in. (AISC 360-05 I3.2d(3)) $R_p = 0.6$

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 $Q_{n\_geom} = R_g \times R_p \times A_{sc} \times F_u$ = 17.2 kips AISC 360-05 (I3-3)

Nominal strength of one stud $Q_n = Min(Q_{n\_conc}, Q_{n\_geom})$ = 17.2 kips

Number of Studs from beam end to maximum moment location N_{studs} = 10

Number of deck ribs from beam end to maximum moment (at beam centerline) N_{ribs} = 16

N_{ribs} > N_{studs} therefore assumption of one stud per rib OK

Horizontal shear at beam-slab interface

Shear in Studs $V_{p\_studs} = N_{studs} \times Q_n$ = **172.3** kips

Shear - Concrete Crushing $V_{p\_concrete\_crushing} = 0.85 \times f_c \times A_c$ = **1606.5** kips

Shear – Steel Yielding $V_{p\_steel\_yield} = F_y \times A_s$ = **515.0** kips

Horizontal shear **V_p =Min(V_{p_studs}, $V_{p\_concrete\_crushing}$, $V_{p\_steel\_yield}$) = 172.3 kips**

Shear at full interaction V_{p_Full} = Min($V_{p\_concrete\_crushing}$, $V_{p\_steel\_yield}$) = **515.0** kips

Percent composite action $Comp_{percent} = V_p / V_{p\_Full}$ = **33.5** %

$Comp_{percent}$ is greater than the absolute minimum (25%) – OK

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

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 beam depth d_s = **17.70** in

Steel beam web thickness t_w = **0.30** in

Steel beam flange thickness t_f = **0.43** in

Area of steel beam web $A_{web} = (d_s - 2 \times t_f) \times t_w$ = **5.06** in^2

Steel beam flange width b_f = **6.00** in

Effective area of each flange for use in composite section calculations

$A_{f\_eff} = (A_s - A_{web}) / 2$ = **2.62** in^2

Effective width of flanges for use in composite section calculations

$b_{f\_eff} = A_{f\_eff} / t_f$ = **6.17** in

Compression force in concrete $C_{conc} = V_p$ = **172.3** kips

Effective depth of concrete in compression $a_{eff} = C_{conc} / (0.85 \times f_c \times b_{eff})$ = **0.48** in

Tensile Strength of steel $P_y = V_{p\_steel\_yield}$ = **515.0** kips

Compression in Steel beam $C_{steel} = (P_y - C_{conc}) / 2$ = **171.4** kips

Max compression force in steel flange $C_{steel\_flange\_max} = F_y \times t_f \times b_{f\_eff}$ = **131.1** kips

$C_{steel} > C_{steel\_flange\_max}$ therefore plastic neutral axis is in the beam web and $C_{steel\_flange} = C_{steel\_flange\_max}$

Compression force in the beam web $C_{steel\_web} = C_{steel} - C_{steel\_flange}$ = **40.2** kips

Length of beam web in compression (below bottom of flange)

$d_{web} = (C_{steel\_web}) / (F_y \times t_w)$ = **2.68** in

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

$PNA = d_{web} + t_f$ = **3.11** 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.

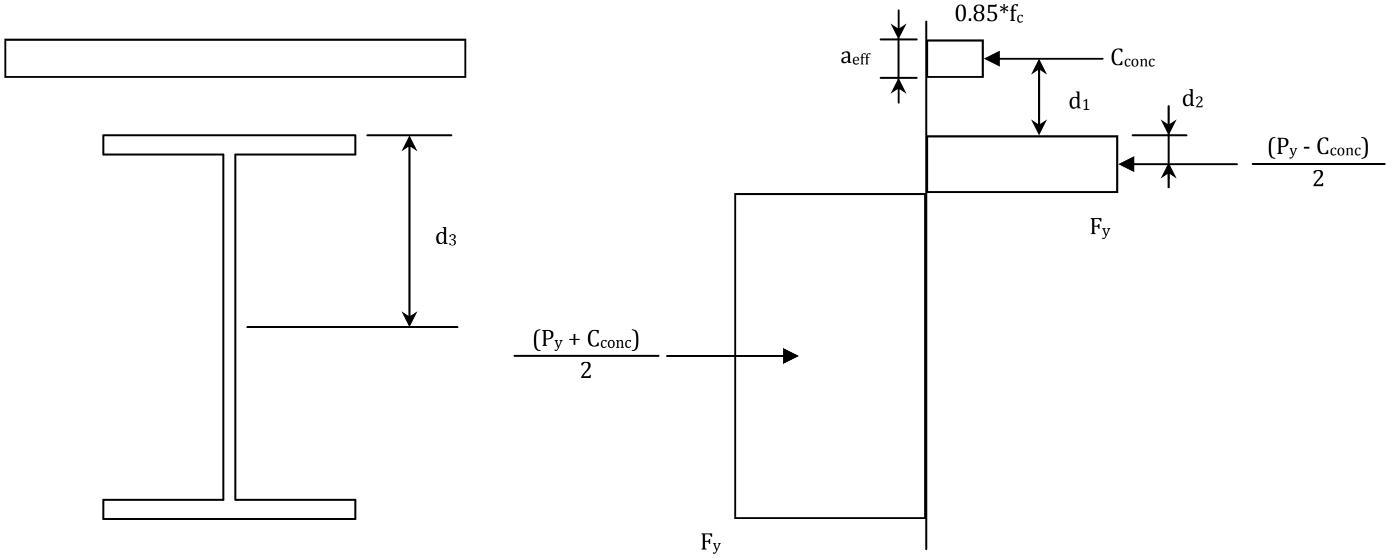


Figure 1: Commentary to the AISC LRFD Specification for Structural Steel Buildings 1999—Fig. C-I3.1: Plastic Stress distribution for positive moment in composite beams.

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

$d_1 = (h_r + t_c) - a_{eff}/2 =$ **6.26** in

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

$$d_2 = \frac{C_{steel\_flange} \times t_f/2 + C_{steel\_web} \times (t_f + d_{web}/2)}{C_{steel}}$$

$d_2 =$ **0.58** in

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

$d_3 = d_s/2 =$ **8.85** in

Nominal Composite Flexural Strength $M_{n\_comp} = C_{conc} \times (d_1 + d_2) + P_y \times (d_3 - d_2) =$ **453.2** kip_ft

Available Composite Flexural Strength $\boldsymbol{M_{c\_comp} = \phi_{b\_comp} \times M_{n\_comp} =}$ **407.9 kip_ft**

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

Composite Stage Design Checks – Elastic Section Properties

Steel Beam Moment of Inertia $I_x =$ **510.0** in^4

Steel Beam Area $A_s =$ **10.30** in^2

Area of Concrete $A_c =$ **472.50** in^2

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

Elastic composite section properties are determined from the configuration in Figure 2, neglecting the contribution of concrete below the top of the metal deck.

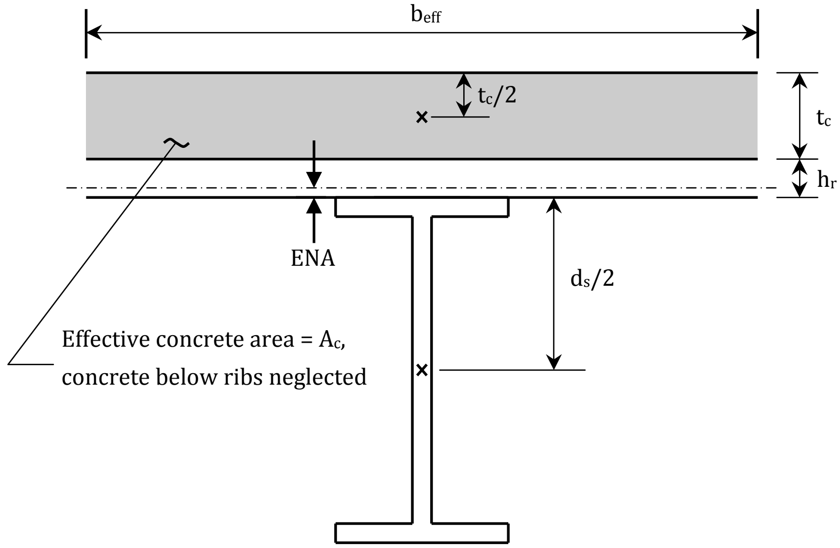


Figure 2: Equivalent Elastic Composite Section

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

$$ENA_{st} = \frac{A_c/n_{st} \times (h_r + t_c/2) - A_s \times d_s/2}{A_c/n_{st} + A_s}$$

ENA_{st} = **2.24** in

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

$$I_{tr\_st} = I_x + A_s\left(d_s/2 + ENA_{st}\right)^2 + \left(b_{eff}/n_{st}\right)t_c^3/12 + \left(A_c/n_{st}\right)\left(t_c/2 + h_r - ENA_{st}\right)^2$$

I_{tr_st} = **2103** in^4

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

$I_{tr\_eff\_st} = 0.75 \times I_{tr\_st}$ = **1577** in^4

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

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

I_{eff_st} = **1127** 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)

$$ENA_{lt} = \frac{A_c/n_{lt} \times (h_r + t_c/2) - A_s \times d_s/2}{A_c/n_{lt} + A_s}$$

ENA_{lt} = **0.77** in

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

$$I_{tr\_lt} = I_x + A_s\left(d_s/2 + ENA_{lt}\right)^2 + \left(b_{eff}/n_{lt}\right)t_c^3/12 + \left(A_c/n_{lt}\right)\left(t_c/2 + h_r - ENA_{lt}\right)^2$$

I_{tr_lt} = **1856** in^4

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

$I_{tr\_eff\_lt} = 0.75 \times I_{tr\_lt}$ = **1392** in^4

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

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

I_{eff_lt} = **1020** in^4

Composite Stage Design Checks – Deflections (Beam Centerline)

Camber = **1.25** in

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

Beam weight — $\Delta_{Beam} = 5 \times (Weight_{BM}) \times L_{bm}^4/(384 \times E_s \times I_x)$ = **0.08** in

Dry slab weight only — $\Delta_{slab\_only} = 5 \times (w_{slab\_dry}) \times L_{bm}^4/(384 \times E_s \times I_x)$ = **1.58** in

Total Slab — $\Delta_{slab\_total} = \Delta_{Beam} + \Delta_{slab\_only}$ = **1.66** in

Slab Adjusted for Camber — $\Delta_{slab} = \Delta_{slab\_total} - Camber$ = **0.41 in**

Slab Deflection Limit — $\Delta_{slab\_comp\_max}$ = **1.75** in

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

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

Short-term live load deflection — $\Delta_{LL\_st} = 5 \times (w_{LL\_st}) \times L_{bm}^4/(384 \times E_s \times I_{eff\_st})$ = **0.69** in

Long-term live load deflection — $\Delta_{LL\_lt} = 5 \times (w_{LL\_lt}) \times L_{bm}^4/(384 \times E_s \times I_{eff\_lt})$ = **0.38** in

Total live load deflection — $\Delta_{LL} = \Delta_{LL\_st} + \Delta_{LL\_lt}$ = **1.07 in**

Live Load Deflection Limit — $\Delta_{LL\_comp\_max}$ = **1.17** 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 — $\Delta_{SDL} = 5 \times (w_{SDL}) \times L_{bm}^4/(384 \times E_s \times I_{eff\_lt})$ = **0.17 in**

Total Deflection

Total Deflection (incl. Camber) — $\Delta_{tot\_comp} = \Delta_{slab} + \Delta_{LL} + \Delta_{SDL}$ = **1.65 in**

Total Deflection Limit — $\Delta_{tot\_comp\_max}$ = **1.75** in

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

For direct comparison with results from composite beam design, the Superimposed Dead load case accounts for the entire 'Dead' deflection given in the results. The self weight deflection reported in FASTRAK Composite Beam Design is adjusted to account for camber. In this case the camber is greater than the self weight deflection. Therefore the self weight deflection is reported as zero. Similarly, the 'slab' deflection from FASTRAK is adjusted for camber and corresponds to Δ_{slab} as reported above.

SUMMARY – W18X35 (20) C=1 ¼"

Construction Stage

Design Condition	*Critical Value*	*Capacity Limit*	*Ratio*
Vertical Shear (End)	V_{r_const} = **21** kips	V_c = **159** kips	V_{r_const} / V_c = **0.134**
Flexure (Centerline)	M_{r_const} = **187** kip_ft	M_{c_const} = **249** kip_ft	M_{r_const} / M_{c_const} = **0.748**
Deflection (Centerline)	Δ_{tot_const} = **0.92** in	$\Delta_{tot\_const\_max}$ = **1.75** in	Δ_{tot_const} / $\Delta_{tot\_const\_max}$ = **0.523**

Composite Stage

Design Condition	*Critical Value*	*Capacity Limit*	*Ratio*
Vertical Shear (End)	V_{r_comp} = **46** kips	V_c = **159** kips	V_{r_comp} / V_c = **0.291**
Flexure (Centerline)	M_{r_comp} = **406** kip_ft	M_{c_comp} = **408** kip_ft	M_{r_comp} / M_{c_comp} = **0.995**
Deflections (Centerline)	Camber = **1.25** in		
Slab (incl. Camber)	Δ_{slab} = **0.41** in	$\Delta_{slab\_comp\_max}$ = **1.75** in	Δ_{slab} / $\Delta_{slab\_comp\_max}$ = **0.232**
Live	Δ_{LL} = **1.07** in	$\Delta_{LL\_comp\_max}$ = **1.17** in	Δ_{LL} / $\Delta_{LL\_comp\_max}$ = **0.916**
Superimposed Dead	Δ_{SDL} = **0.17** in	NA	
Total	Δ_{tot_comp} = **1.65** in	$\Delta_{tot\_comp\_max}$ = **1.75** in	Δ_{tot_comp} / $\Delta_{tot\_comp\_max}$ = **0.941**

DESIGN METHOD:

There is a direct relationship between the safety factors (Ω) used in ASD and the resistance factors (ϕ) used in LRFD. Namely, Ω=1.5/ϕ. 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: $(w_{LL})/(w_{SDL} + w_{slab\_dry} + Weight\_{BM})$ = **1.14**

This means there is the potential that the ASD method will require a heavier steel section or more studs. In fact, the ASD design for this example requires 26 studs instead of 20. The details of the ASD design are presented in the design example entitled "ASD Beam" – available on the online support website: http://www.cscworld.com/fastrak/us/composite_resources.html.