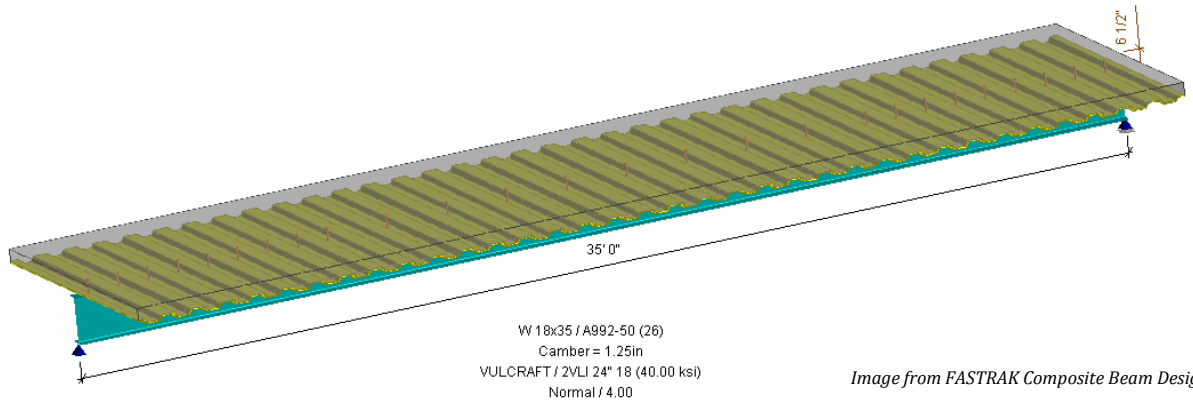


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FASTRAK Composite Beam Design

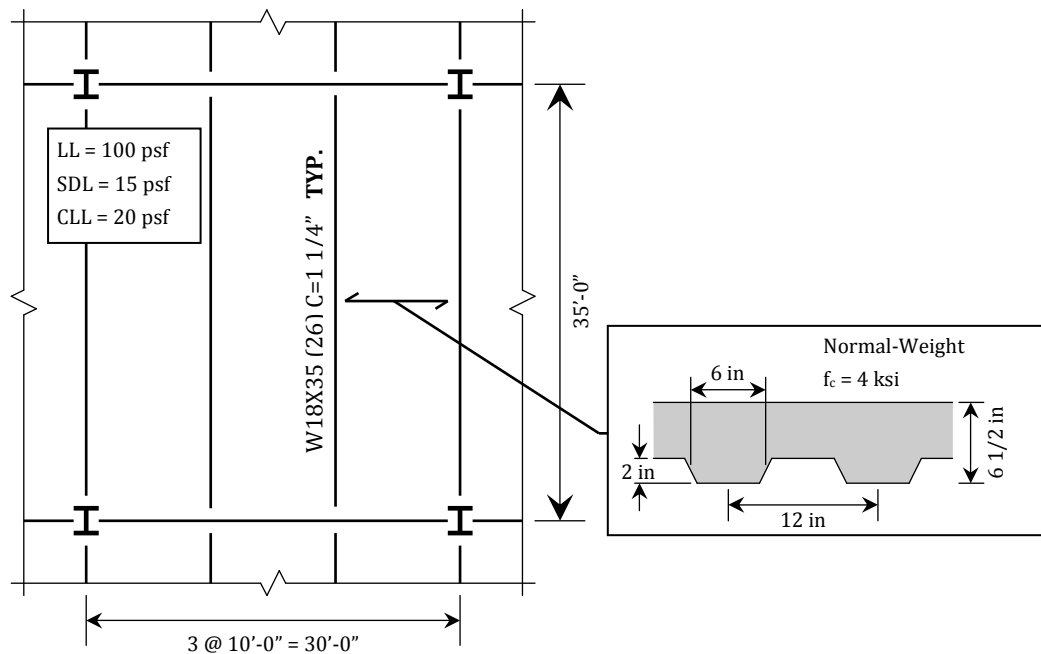
ASD Beam Design Example Calculation

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



The purpose of this document is to help you quickly build confidence when using FASTRAK. This document shows the long-hand engineering for the ASD 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





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BASIC DATA

Typical Interior Beam: W18X35 (26) 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	$\rho_{LL,lt} = 33\%$
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	$\rho_{Ec} = 0.5$
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 safety factor – steel section	$\Omega_{b,steel} = 1.67$	AISC 360-05 F1.1
Bending safety factor – composite section	$\Omega_{b,comp} = 1.67$	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 3/4 in of camber at 1/4 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

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Studs

Distance from stud to deck $e_{mid-ht} < 2$ 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 $Weight_{BM} = 35.0$ plf
 Slab and Deck
 Wet Slab $W_{slab_wet} = (F_{c_wet} + F_{md}) \times S_{bm} = 714$ plf
 Dry Slab $W_{slab_dry} = (F_{c_dry} + F_{md}) \times S_{bm} = 691$ plf
 Live
 Full $w_{LL} = F_{LL} \times S_{bm} = 1000$ plf
 Long-term $w_{LL_lt} = F_{LL_lt} \times S_{bm} = 330.0$ plf
 Short-term $w_{LL_st} = F_{LL_st} \times S_{bm} = 670$ plf
 Superimposed Dead Load $w_{SDL} = F_{SDL} \times S_{bm} = 150$ plf
 Construction Live Load $w_{CLL} = F_{CLL} \times S_{bm} = 200$ plf

Design Loads (ASD)

Dead Load strength combination factor $f_{DL_st} = 1.0$
 Live Load strength combination factor $f_{LL_st} = 1.0$
 Construction Stage Line Load (uses wet slab weight) $w_{r_const} = f_{DL_st} \times (Weight_{BM} + W_{slab_wet}) + f_{LL_st} \times (w_{CLL}) = 949$ plf
 Composite Stage Line Load (uses dry slab weight) $w_{r_comp} = f_{DL_st} \times (Weight_{BM} + W_{slab_dry} + w_{SDL}) + f_{LL_st} \times (w_{LL}) = 1876$ plf

CONSTRUCTION STAGE

Construction Stage Design Checks – Shear (Beam End)

Required Shear Strength $V_{r_const} = w_{r_const} \times (L_{bm}/2) = 16.6$ kips
 Web slenderness ratio $h_{to_tw} = 53.5$
 Compact web maximum slenderness ratio $h_{to_tw_max} = 2.24 \times \sqrt{E_s/F_y} = 53.9$
 $h_{to_tw} < h_{to_tw_max}$ therefore AISC 360-05 G2.1(a) and (G2-2) apply and $C_v = 1.0$
 Shear safety factor – steel only $\Omega_{v_steel} = 1.50$
 Web area $A_w = 5.31$ in²
 Nominal shear strength $V_n = 0.6 \times F_y \times A_w \times C_v = 159.3$ kips (G2-1)
Available shear strength $V_c = V_n / \Omega_{v_steel} = 106.2$ kips
 $V_c > V_{r_const}$ therefore construction stage shear strength is OK

Construction Stage Design Checks – Flexure (Beam Centerline)

Required flexural strength $M_{r_const} = w_{r_const} \times (L_{bm}^2/8) = 145.3$ 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 $Z_x = 66.5$ in³
 Nominal Flexural Strength $M_{n_const} = F_y \times Z_x = 277.1$ kip_ft (F2-1)



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Available Flexural Strength

$$M_{c_const} = M_{n_const} / \Omega_{b_steel} = 165.9 \text{ kip_ft}$$

$M_{c_const} > M_{r_const}$ therefore construction stage flexural strength is OK

Construction Stage Design Checks - Deflection (Beam Centerline)

Moment of Inertia of bare steel beam	$I_x = 510.0 \text{ in}^4$
Dead Load deflection - due to beam self weight and slab wet (includes metal deck weight)	
Dead load Deflection	$\Delta_{DL_const} = 5 \times (W_{slab_wet} + Weight_{BM}) \times L_{bm}^4 / (384 \times E_s \times I_x) = 1.71 \text{ in}$
Camber	$0.75 \times \Delta_{DL_const} = 1.28 \text{ in}$ - therefore Camber = 1.25 in
Construction Live load deflection	$\Delta_{LL_const} = 5 \times (W_{CLL}) \times L_{bm}^4 / (384 \times E_s \times I_x) = 0.46 \text{ in}$
Total construction stage deflection	$\Delta_{tot_const} = (\Delta_{DL_const} - \text{Camber}) + \Delta_{LL_const} = 0.92 \text{ in}$
Construction Stage Deflection Limit	$\Delta_{tot_const_max} = 1.75 \text{ in}$

$\Delta_{tot_const_max} > \Delta_{tot_const}$ therefore construction stage deflection OK

COMPOSITE STAGE

Composite Stage Design Checks - Shear (Beam End)

Required Shear Strength

$$V_{r_comp} = w_{r_comp} \times (L_{bm}/2) = 32.8 \text{ kips}$$

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 $V_n = 159.3 \text{ kips}$ (G2-1)

Available shear strength

$$V_c = V_n / \Omega_{v_steel} = 106.2 \text{ kips}$$

$V_c > V_{r_comp}$ therefore shear strength is OK

Composite Stage Design Checks - Flexure (Beam Centerline)

Required flexural strength

$$M_{r_comp} = w_{r_comp} \times (L_{bm}^2/8) = 287.2 \text{ kip_ft}$$

Method to Determine Nominal Flexural Strength

Web slenderness ratio $h_{to_tw} = 53.5$

Web maximum slenderness ratio $h_{to_tw_maxcomp} = 3.76 \times \sqrt{(E_s/F_y)} = 90.6$

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

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

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 $A_s = 10.3 \text{ in}^2$

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 $e_{mid-h} < 2 \text{ in}$. (AISC 360-05 I3.2d(3)) $R_p = 0.6$

Nominal Stud Strength

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

Nominal strength based on concrete $Q_{n_conc} = 0.5 \times A_{sc} \times \sqrt{(f_c \times E_{c_st})} = 26.1 \text{ kips}$ AISC 360-05 (I3-3)

Nominal strength based on geometry $Q_{n_geom} = R_g \times R_p \times A_{sc} \times F_u = 17.2 \text{ kips}$ AISC 360-05 (I3-3)

Nominal strength of one stud $Q_n = \text{Min}(Q_{n_conc}, Q_{n_geom}) = 17.2 \text{ kips}$

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Number of Studs from beam end to maximum moment location $N_{studs} = 13$

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	$V_{p_studs} = N_{studs} \times Q_n = 224.0$ 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	$V_p = \text{Min}(V_{p_studs}, V_{p_concrete_crushing}, V_{p_steel_yield}) = 224.0$ kips
Shear at full interaction	$V_{p_Full} = \text{Min}(V_{p_concrete_crushing}, V_{p_steel_yield}) = 515.0$ kips
Percent composite action	$\text{Comp}_{percent} = V_p / V_{p_Full} = 43.5$ %

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	$t_r = 0.43$ in
Area of steel beam web	$A_{web} = (d_s - 2 \times t_r) \times t_w = 5.06$ 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	$b_{f_eff} = A_{f_eff} / t_r = 6.17$ in
Compression force in concrete	$C_{conc} = V_p = 224.0$ kips
Effective depth of concrete in compression	$a_{eff} = C_{conc} / (0.85 \times f_c \times b_{eff}) = 0.63$ in
Tensile Strength of steel	$P_y = V_{p_steel_yield} = 515.0$ kips
Compression in Steel beam	$C_{steel} = (P_y - C_{conc}) / 2 = 145.5$ kips
Max compression force in steel flange	$C_{steel_flange_max} = F_y \times t_r \times b_{f_eff} = 131.1$ 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	$C_{steel_web} = C_{steel} - C_{steel_flange} = 14.4$ kips
Length of beam web in compression (below bottom of flange)	$d_{web} = (C_{steel_web}) / (F_y \times t_w) = 0.96$ in
Distance (down) of location of plastic neutral axis from top of steel beam	$PNA = d_{web} + t_r = 1.38$ in

Nominal Moment Strength is determined using Figure C-13.1 (shown below) and Equation(C-13-5) from the Commentary to AISC LRFD Specification for Structural Steel Buildings 1999. See Figure 1.

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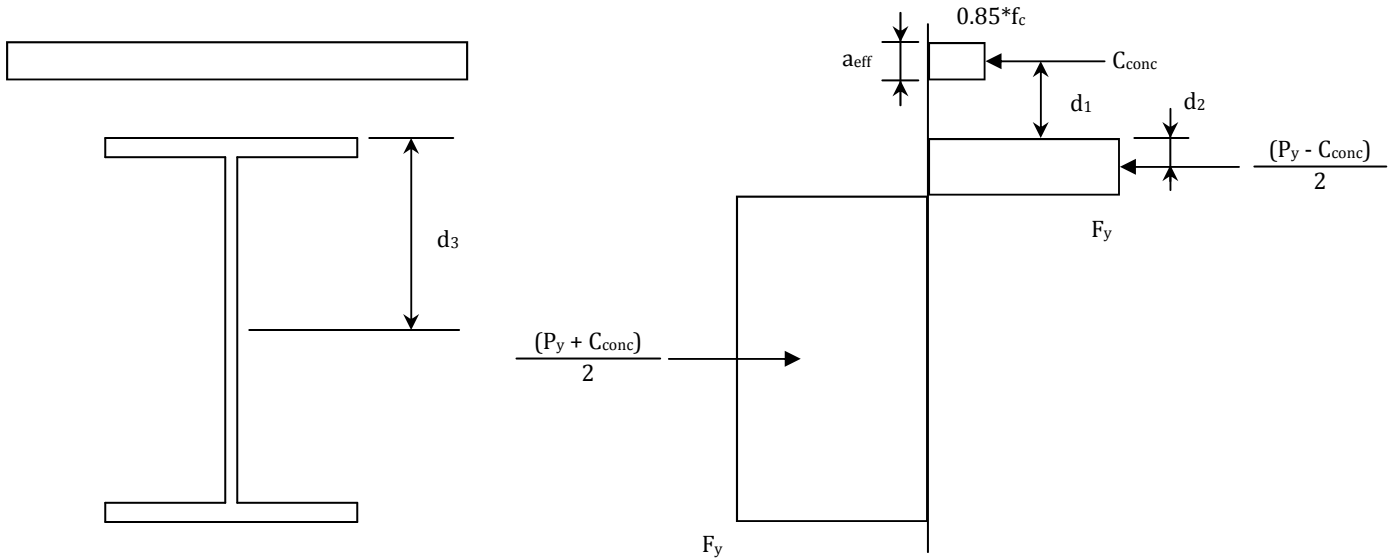


Figure 1: Commentary to the AISC LRFD Specification for Structural Steel Buildings 1999—Fig. C-I3.1: Plastic Stress distribution for positive moment in composite beams.

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

$$d_1 = (h_r + t_c) - a_{eff}/2 = \mathbf{6.19 \text{ in}}$$

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

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

$$d_2 = \mathbf{0.28 \text{ in}}$$

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

$$d_3 = d_s/2 = \mathbf{8.85 \text{ in}}$$

Nominal Composite Flexural Strength

$$M_{n_comp} = C_{conc} \times (d_1 + d_2) + P_y \times (d_3 - d_2) = \mathbf{488.5 \text{ kip_ft}}$$

Available Composite Flexural Strength

$$M_{c_comp} = M_{n_comp} / \Omega_{b_comp} = \mathbf{292.5 \text{ kip_ft}}$$

$M_{c_comp} > M_{r_comp}$ therefore shear strength is OK

Composite Stage Design Checks – Elastic Section Properties

Steel Beam Moment of Inertia

$$I_x = \mathbf{510.0 \text{ in}^4}$$

Steel Beam Area

$$A_s = \mathbf{10.30 \text{ in}^2}$$

Area of Concrete

$$A_c = \mathbf{472.50 \text{ in}^2}$$

Short-term modular ratio

$$n_{st} = E_s/E_{c_st} = \mathbf{8.3}$$

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

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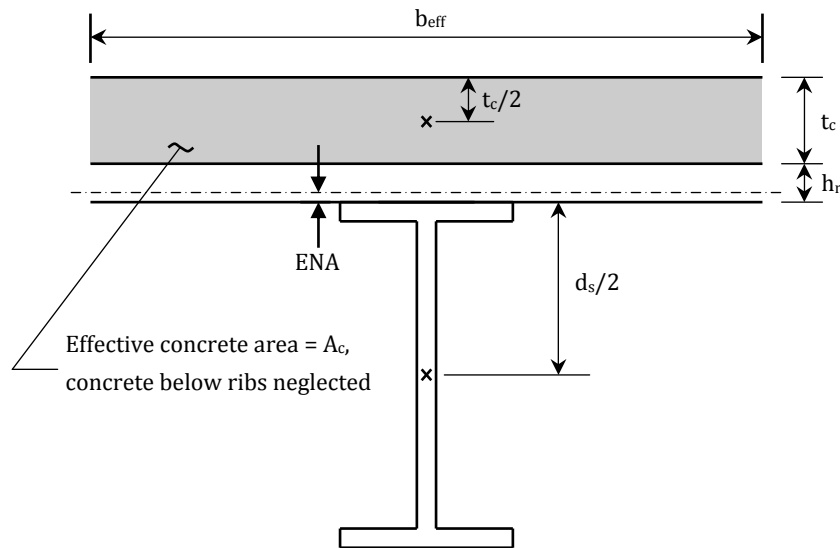


Figure 2: Equivalent Elastic Composite Section

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

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

$I_{tr_st} = 2103$ in⁴

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

$I_{tr_eff_st} = 0.75 \times I_{tr_st} = 1577$ in⁴

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

$$I_{eff_st} = I_x + (I_{tr_eff_st} - I_x) \sqrt{V_p/V_{p_Full}}$$

$I_{eff_st} = 1214$ in⁴

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

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

$I_{tr_lt} = 1856$ in⁴

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

$I_{tr_eff_lt} = 0.75 \times I_{tr_lt} = 1392$ in⁴



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Long-term effective moment of inertia due to partial composite action AISC 360-05 Commentary (C-13-3), V_p at centerline

$$I_{eff_lt} = I_x + (I_{tr_eff_lt} - I_x) \sqrt{V_p / V_{p_Full}}$$

$$I_{eff_lt} = 1092 \text{ in}^4$$

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 $\Delta_{Beam} = 5 \times (\text{Weight}_{BM}) \times L_{bm}^4 / (384 \times E_s \times I_x) = 0.08 \text{ in}$

Dry slab weight only $\Delta_{slab_only} = 5 \times (w_{slab_dry}) \times L_{bm}^4 / (384 \times E_s \times I_x) = 1.58 \text{ in}$

Total Slab $\Delta_{slab_total} = \Delta_{Beam} + \Delta_{slab_only} = 1.66 \text{ in}$

Slab Adjusted for Camber $\Delta_{slab} = \Delta_{slab_total} - \text{Camber} = 0.41 \text{ in}$

Slab Deflection Limit $\Delta_{slab_comp_max} = 1.75 \text{ in}$

$\Delta_{slab_comp_max} > \Delta_{slab}$ 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 $\Delta_{LL_st} = 5 \times (w_{LL_st}) \times L_{bm}^4 / (384 \times E_s \times I_{eff_st}) = 0.64 \text{ in}$

Long-term live load deflection $\Delta_{LL_lt} = 5 \times (w_{LL_lt}) \times L_{bm}^4 / (384 \times E_s \times I_{eff_lt}) = 0.35 \text{ in}$

Total live load deflection $\Delta_{LL} = \Delta_{LL_st} + \Delta_{LL_lt} = 0.99 \text{ in}$

Live Load Deflection Limit $\Delta_{LL_comp_max} = 1.17 \text{ in}$

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

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

Superimposed Dead $\Delta_{SDL} = 5 \times (w_{SDL}) \times L_{bm}^4 / (384 \times E_s \times I_{eff_lt}) = 0.16 \text{ in}$

Total Deflection

Total Deflection (incl. Camber) $\Delta_{tot_comp} = \Delta_{slab} + \Delta_{LL} + \Delta_{SDL} = 1.56 \text{ in}$

Total Deflection Limit $\Delta_{tot_comp_max} = 1.75 \text{ in}$

$\Delta_{tot_comp_max} > \Delta_{tot_comp}$ 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.



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SUMMARY - W18X35 (26) C=1 1/4"

Construction Stage

<i>Design Condition</i>	<i>Critical Value</i>	<i>Capacity Limit</i>	<i>Ratio</i>
Vertical Shear (End)	$V_{r_const} = 17$ kips	$V_c = 106$ kips	$V_{r_const} / V_c = 0.156$
Flexure (Centerline)	$M_{r_const} = 145$ kip_ft	$M_{c_const} = 166$ kip_ft	$M_{r_const} / M_{c_const} = 0.875$
Deflection (Centerline)	$\Delta_{tot_const} = 0.92$ in	$\Delta_{tot_const_max} = 1.75$ in	$\Delta_{tot_const} / \Delta_{tot_const_max} = 0.523$

Composite Stage

<i>Design Condition</i>	<i>Critical Value</i>	<i>Capacity Limit</i>	<i>Ratio</i>
Vertical Shear (End)	$V_{r_comp} = 33$ kips	$V_c = 106$ kips	$V_{r_comp} / V_c = 0.309$
Flexure (Centerline)	$M_{r_comp} = 287$ kip_ft	$M_{c_comp} = 292$ kip_ft	$M_{r_comp} / M_{c_comp} = 0.982$
Deflections (Centerline)	Camber = 1.25 in		
Slab (incl. Camber)	$\Delta_{slab} = 0.41$ in	$\Delta_{slab_comp_max} = 1.75$ in	$\Delta_{slab} / \Delta_{slab_comp_max} = 0.232$
Live	$\Delta_{LL} = 0.99$ in	$\Delta_{LL_comp_max} = 1.17$ in	$\Delta_{LL} / \Delta_{LL_comp_max} = 0.853$
Superimposed Dead	$\Delta_{SDL} = 0.16$ in	NA	
Total	$\Delta_{tot_comp} = 1.56$ in	$\Delta_{tot_comp_max} = 1.75$ in	$\Delta_{tot_comp} / \Delta_{tot_comp_max} = 0.892$

DESIGN METHOD:

There is a direct relationship between the safety factors (Ω) used in ASD and the resistance factors (ϕ) used in 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: $(w_{LL})/(w_{SDL} + w_{slab_dry} + Weight_{BM}) = 1.14$

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