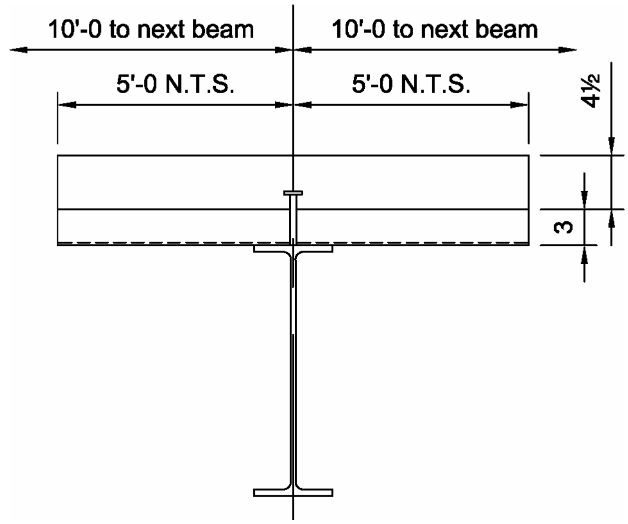
ASDIP Steel
STRUCTURAL ENGINEERING SOFTWARE

Example I-1 Composite Beam Design

Given:

A series of 45-ft. span composite beams at 10 ft. o/c are carrying the loads shown below. The beams are ASTM A992 and are unshored. The concrete has f'_c = 4 ksi. Design a typical floor beam with 3 in. 18 gage composite deck, and 4½ in. normal weight concrete above the deck, for fire protection and mass. Select an appropriate beam and determine the required number of shear studs.



Solution:

Material Properties:

Concrete f'_c = 4 ksi

Beam F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Loads:

Dead load:

Slab = 0.075 kip/ft^2
Beam weight = 0.008 kip/ft^2 (assumed)
Miscellaneous = 0.010 kip/ft^2 (ceiling etc.)

Live load:

Non-reduced = 0.10 kips/ft^2

Since each beam is spaced at 10 ft. o.c.

Total dead load = 0.093 kip/ft^2(10 ft.) = 0.93 kips/ft.
Total live load = 0.10 kip/ft^2(10ft.) = 1.00 kips/ft.

Construction dead load (unshored) = 0.083 kip/ft^2(10 ft) = 0.83 kips/ft
Construction live load (unshored) = 0.020 kip/ft^2(10 ft) = 0.20 kips/ft

Determine the required flexural strength

LRFD	ASD
w_u = 1.2(0.93 kip/ft) + 1.6(1.0 kip/ft) = 2.72 kip/ft $M_u = \dfrac{2.72 \text{ kip/ft}(45 \text{ ft})^2}{8}$ = 687 kip-ft.	w_a = 0.93 kip/ft + 1.0 kip/ft = 1.93 kip/ft $M_a = \dfrac{1.93 \text{ kip/ft}(45 \text{ ft})^2}{8}$ = 489 kip-ft.

Use Tables 3-19, 3-20 and 3-21 from the Manual to select an appropriate member

Determine b_{eff}

Section I.3.1.1a

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:

(1) one-eighth of the beam span, center to center of supports

$$\frac{45 \text{ ft.}}{8}(2) = 11.3 \text{ ft.}$$

(2) one-half the distance to center-line of the adjacent beam

$$\frac{10 \text{ ft.}}{2}(2) = 10.0 \text{ ft.}$$ **Controls**

(3) the distance to the edge of the slab
Not applicable

Calculate the moment arm for the concrete force measured from the top of the steel shape, Y2.

Assume a = 1.0 in. (Some assumption must be made to start the design process. An assumption of 1.0 in. has proven to be a reasonable starting point in many design problems.)

$Y2 = t_{slab} - a/2 = 7.5 - ½ = 7.0$ in.

Enter Manual Table 3-19 with the required strength and $Y2$=7.0 in. Select a beam and neutral axis location that indicates sufficient available strength.

Manual Table 3-19

Select a W21×50 as a trial beam.

When PNA location 5 (BFL), this composite shape has an available strength of:

LRFD	ASD
$\phi_b M_n$= 770 kip-ft > 687 kip-ft **o.k.**	M_n/Ω_b = 512 kip-ft > 489 kip-ft **o.k.**

Manual Table 3-19

Note that the required PNA location for ASD and LRFD differ. This is because the live to dead load ratio in this example is not equal to 3. Thus, the PNA location requiring the most shear transfer is selected to be acceptable for ASD. It will be conservative for LRFD.

Check the beam deflections and available strength

Check the deflection of the beam under construction, considering only the weight of concrete as contributing to the construction dead load.

Limit deflection to a maximum of 2.5 in. to facilitate concrete placement.

$$I_{req} = \frac{5}{384}\frac{w_{DL}l^4}{E\Delta} = \frac{5(0.83 \text{ kip/ft})(45 \text{ ft})^4(1728 \text{ in.}^3/\text{ft}^3)}{384(29{,}000 \text{ ksi})(2.5 \text{ in.})} = 1{,}060 \text{ in.}^4$$

From Manual Table 3-20, a W21×50 has I_x = 984 in.^{4}, therefore this member does not satisfy the deflection criteria under construction.

Using Manual Table 3-20, revise the trial member selection to a W21×55, which has I_x = 1140 in.^{4}, as noted in parenthesis below the shape designation.

Check selected member strength as an un-shored beam under construction loads assuming adequate lateral bracing through the deck attachment to the beam flange.

LRFD	ASD
Calculate the required strength 1.4 DL = 1.4 (0.83 kips/ft) = 1.16 kips/ft 1.2DL+1.6LL = 1.2 (0.83) + 1.6(0.20) = 1.32 klf $M_{u\ (\text{unshored})} = \dfrac{1.31 \text{ kip/ft}(45 \text{ ft})^2}{8}$ =331 kip-ft The design strength for a W21×55 is 473 kip-ft > 331 kip-ft **o.k.**	*Calculate the required strength* DL+LL = 0.83 + 0.20 = 1.03 kips.ft $M_{a\ (\text{unshored})} = \dfrac{1.03 \text{ kips/ft}(45 \text{ ft})^2}{8}$ = 260 kip-ft The allowable strength for a W21× 55 is 314 kip-ft > 260 kip-ft **o.k.**

For a W21×55 with $Y2$=7.0 in, the member has sufficient available strength when the PNA is at location 6 and $\sum Q_n$ = 292 kips. Manual Table 3-19

LRFD	ASD
$\phi_b M_n$=767 kip-ft > 687 kip-ft **o.k.**	M_n/Ω_b = 510 kip-ft > 489 kip-ft **o.k.**

Manual Table 3-19

Check a

$$a = \frac{\sum Q_n}{0.85 f_c' b} = \frac{292 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft.})(12 \text{ in./ft.})} = 0.716 \text{ in.}$$

0.716 in. < 1.0 in. assumed **o.k.**

Check live load deflection

$\Delta_{LL} < l/360$ = ((45 ft.)(12 in./ft))/360 = 1.5 in.

A lower bound moment of inertia for composite beams is tabulated in Manual Table 3-20.

For a W21×55 with $Y2$=7.0 and the PNA at location 6, I_{LB} = 2440 in.^{4} — Manual Table 3-20

$$\Delta_{LL} = \frac{5}{384}\frac{w_{LL}l^4}{EI_{LB}} = \frac{5\left(1.0\text{ kip/ft}\right)\left(45\text{ ft}\right)^4\left(1728\text{ in.}^3/\text{ft}^3\right)}{384\left(29{,}000\text{ ksi}\right)\left(2440\text{ in.}^4\right)} = 1.30\text{ in.}$$

1.30 in. < 1.5 in. **o.k.**

Determine if the beam has sufficient available shear strength — Manual Table 3-3

LRFD	ASD
$V_u = \frac{45\text{ft}}{2}\left(2.72\text{kip/ft}\right) = 61.2\text{ kips}$ ϕV_n= 234 kips > 61.2 kips **o.k.**	$V_a = \frac{45\text{ft}}{2}\left(1.93\text{kip/ft}\right) = 43.4\text{ kips}$ V_n/Ω = 156 kips > 43.4 kips **o.k.**

Determine the required number of shear stud connectors

Using perpendicular deck with one ¾-in. diameter weak stud per rib in normal weight 4 ksi concrete. Q_n = 17.2 kips/stud — Manual Table 3-21

$$\frac{\sum Q_n}{Q_n} = \frac{292\text{ kips}}{17.2\text{ kips}} = 17\text{, on each side of the beam.}$$

Section 3.2d(5)

Total number of shear connectors; use 2(17) = 34 shear connectors. — Section 3.2d(6)

Check the spacing of shear connectors

Since each flute is 12 in., use one stud every flute, starting at each support, and proceed for 17 studs on each end of the span.

$6d_{stud}$ < 12 in. < $8t_{slab}$, therefore, the shear stud spacing requirements are met. — Section I3.2c (b)

The studs are to be 5 in. long, so that they will extend a minimum of 1½ in. into slab.

Project:
Engineer:
Descrip: Composite Beam Verification

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ASDIP Steel 2.2.0 — **STEEL BEAM DESIGN** — www.asdipsoft.com

GEOMETRY

Column Designation	W21X55		
Steel Yield Strength Fy	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	45.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb	0.00	ft	OK

PROPERTIES

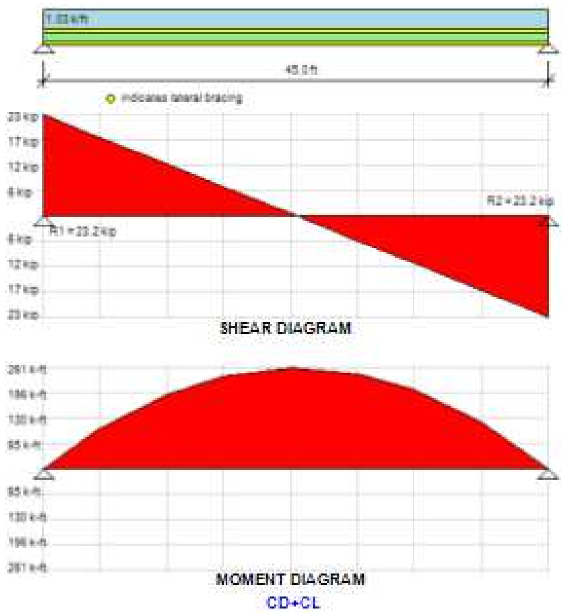
Area ..	16.2	in2	Sx ...	110.0	in3
Depth	20.8	in	Zx ...	126.0	in3
bf	8.2	in	rx	8.40	in
tw	0.38	in	Iy	48.4	in4
tf	0.52	in	Sy ...	11.8	in3
k des .	1.02	in	Zy ...	18.4	in3
Ix	1140.0	in4	ry	1.73	in

SERVICE LOADS (ASD)

	Uniform (k/ft)		Concentrated (kip)					
	w1	w2	P1	P2	P3	P4	P5	P6
Dead Load	0.93	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Live Load	1.00	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Const. Dead Load ..	0.83	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Const. Live Load	0.20	0.00	0.0	0.0	0.0	0.0	0.0	0.0
Start Distance (ft) ...	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
End Distance (ft)	45.00	0.00						

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00		
Maximum Shear Force V ..	43.4	kip	
Limit States	Nominal Vn		
Shear Yielding	234.0	kip	✓
Shear Buckling	234.0	kip	
Nominal Strength Vn	234.0	kip	
Safety Factor Ω	1.50		
Allowable Strength Vn/Ω .	156.0	kip	
V / Vn/Ω Design Ratio	0.28		OK



LOCAL BUCKLING

Flexure in Flanges	Compact
Compression in Flanges	Non-compact
Flexure in Web	Compact
Compression in Web	Slender

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.14		
Max. Bending Moment M ..	260.8	k-ft	
Limit States	**Nominal Mn**		
Yielding	525.0	k-ft	✓
Lateral Torsional Buckling	525.0	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	
Nominal Strength Mn	525.0	k-ft	
Safety Factor Ω	1.67		
Allowable Strength Mn/Ω .	314.4	k-ft	
M / Mn/Ω Design Ratio	0.83		OK

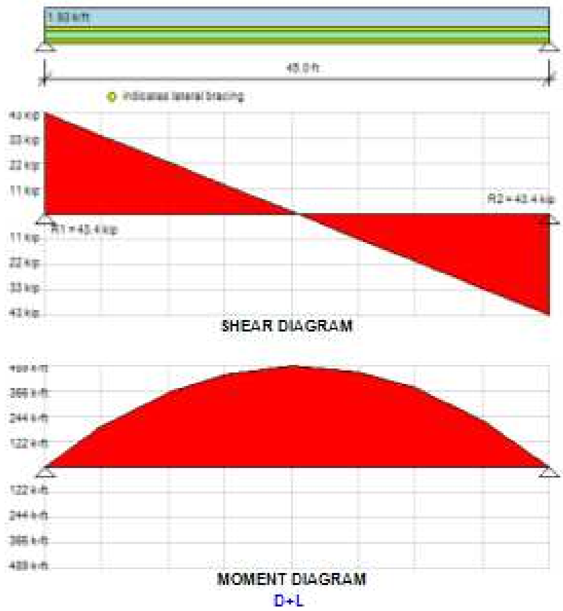
FLEXURE DESIGN (COMPOSITE)

Overall Slab Thickness	7.5	in	OK
Interior Beam. Spacing = 10.0 ft			
Effective Slab Width	10.00	ft	
Concrete Strength f'c	4000	psi	OK
Concrete Density	150	pcf	OK
Metal Deck Type	VULCRAFT 3 VLI		
Deck Ribs Height hr	3.0	in	OK
Deck Ribs Avg. Width wr ...	6.0	in	OK
Deck Ribs Run Perpendicular to the Beam			
Max. Bending Moment M ..	488.5	k-ft	
Limit States	**Nominal Mn**		
Plastic Yielding	850.4	k-ft	✓
Elastic Yielding	N.A.	k-ft	
Nominal Strength Mn	850.4	k-ft	
Safety Factor Ω	1.67		
Allowable Strength Mn/Ω .	509.2	k-ft	
M / Mn/Ω Design Ratio	0.96		OK

DEFLECTIONS

Required Camber	2.29	in
Long-term Deflection	0.62	in

Loading	delta	L/delta	Ratio	
CL	0.56 in	967	0.37	OK
CD+CL .	0.58 in	929	0.26	OK
L	1.27 in	424	0.85	OK
D+L	2.04 in	264	0.91	OK



SHEAR CONNECTORS

Shear Stud Diameter	3/4"		OK
Shear Stud Length	5.0	in	OK
Tensile Strength Fu	65.0	ksi	
Nominal Strength Qn	17.2	kip	
Horizontal Shear Force	283.5	kip	
# of Studs for Full Composite	96		
# of Studs for Partial Composite .	34		OK
Partial Composite Action %	35	%	OK
Minimum Spacing Allowed ..	4.5	in	
# of Studs at Any Section	1		
Max. Spacing Required	15.9	in	OK