9.25 MATERIAL PROPERTIES:

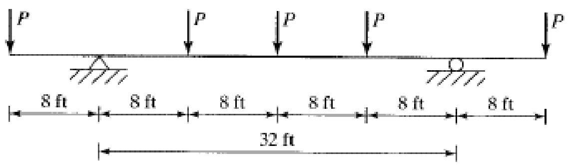
Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_{y} =	50	ksi
Ultimate Strength	Fu =	70	ksi

Reference: AISC 14th

Section	Eq/Fig/Table
F	

PROBLEM 9.25

9-24. A beam of F_y = 50 ksi steel is used to support the loads shown in Fig. P9-24. Neglecting the beam self-weight, determine the lightest W shape to carry the loads if full lateral bracing is provided.



P: P_D = 8.5 k, P_L = 6.0 k

FIGURE P9-24

9-25. Redesign the beam of Prob. 9-24 if lateral bracing is only provided at the supports and at the concentrated loads. Determine C_b. (*Ans.* W16 × 26 LRFD, W14 × 30 ASD)

Required:

a) Design Beam of Figure P9-24

Method:

i) Determine Beam Flexural Demand
ii) Determine Moment Distribution on the beam
iii) Determine Cb
iv) Determine Beam Flexural Capacity

Solution:

Member Length	L =	38	ft	L =	38	ft
Number of Point Loads		5			5	
Number of Supports		2			2	
Dead Load	DL=	8.5	kip	DL=	8.5	kip
Live Load	LL=	6	kip	LL=	6	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
Unbraced Length	L_{bx} =	8	ft	L_{bx} =	8	ft

Reference: AISC 14th

Section	Eq/Fig/Table
F	

LRFD | **ASD**

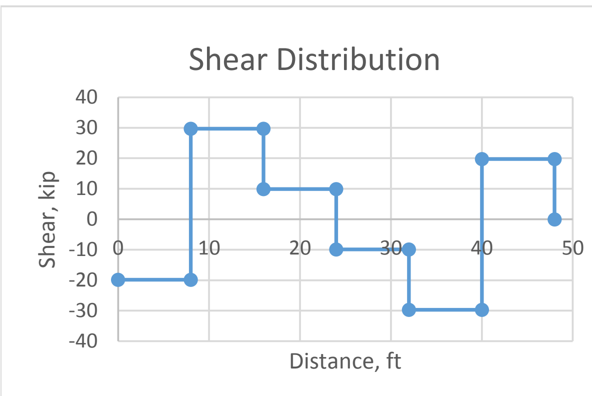
1) Demand:				**Demand:**			
Load	Pu =	19.8	kip		Pa =	14.5	0
Reactions	Ry =	49.5	kip	Reactions	Ry =	36.25	0
	Rx =	0	kip		Rx =	0	0

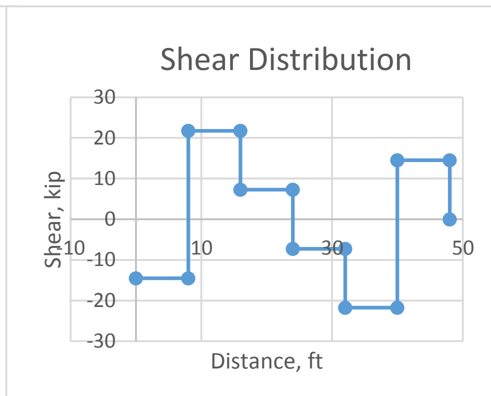
Shear Distribution:

Load (#)	Load Value (kip)	Distance (ft)	Shear (kip)
1	-19.8	0	-19.8
Ra	49.5	8	29.7
2	-19.8	16	9.9
3	-19.8	24	-9.9
4	-19.8	32	-29.7
Rb	49.5	40	19.8
5	-19.8	48	0

Shear Distribution:

Load (#)	Load Value (kip)	d (ft)	Shear (kip)
1	-14.5	0	-14.5
Ra	36.25	8	21.75
2	-14.5	16	7.25
3	-14.5	24	-7.25
4	-14.5	32	-21.75
Rb	36.25	40	14.5
5	-14.5	48	0



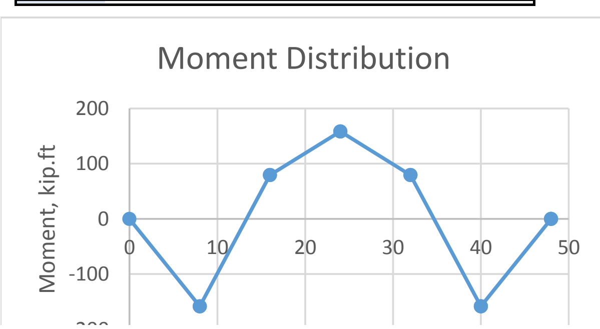


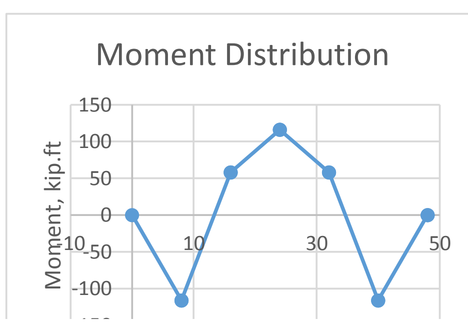
Moment Distribution:

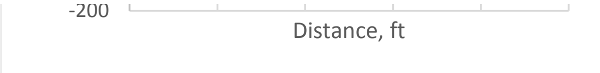
Load (#)	Shear (kip)	Distance (ft)	Moment (kip.ft)
1	-19.8	0	0
Ra	29.7	8	-158.4
2	9.9	16	79.2
3	-9.9	24	158.4
4	-29.7	32	79.2
Rb	19.8	40	-158.4
5	0	48	0

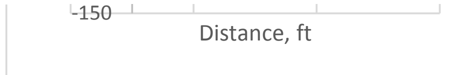
Moment Distribution:

Load (#)	Shear (kip)	Distance (ft)	Moment (kip.ft)
1	-14.5	0	0
Ra	21.75	8	-116
2	7.25	16	58
3	-7.25	24	116
4	-21.75	32	58
Rb	14.5	40	-116
5	0	48	0









ii) Determine Cb (LRFD)

M_{max} (kip.ft)	$M_{.25}$ (kip.ft)	$M_{.5}$ (kip.ft)	$M_{.75}$ (kip.ft)	C_b
158.4	39.6	79.2	118.8	1.67
158.4	99	39.6	19.8	2.17
158.40	99	118.8	138.6	1.25
158.4	19.8	39.6	99	2.17
158.4	118.8	79.2	39.6	1.67

ii) Determine Cb (ASD)

M_{max} (kip.ft)	$M_{.25}$ (kip.ft)	$M_{.5}$ (kip.ft)	$M_{.75}$ (kip.ft)	C_b
116	29	58	87	1.67
116	72.5	29	14.5	2.17
116	72.5	87	101.5	1.25
116	14.5	29	72.5	2.17
116	87	58	29	1.67

	LRFD			ASD			
USE	$C_b =$	1.25		$C_b =$	1.25		**ANSWER**
Maximum Moment	$M_{max} =$	158.4	kip.ft	$M_{max} =$	116	kip.ft	
Effective Moment	$M_{ueff} =$	126.72	kip.ft	$M_{ueff} =$	92.8	kip.ft	
Unbraced Length	$L_{bx} =$	8	ft	$L_{bx} =$	8	ft	
Beam Selection		**W16X26**			**W14X30**		Table 3-2
Full plastic yield Lengtl	$L_p =$	3.96	ft	$L_p =$	5.26	ft	Table 3-2
LTB Length:	$L_r =$	11.2	ft	$L_r =$	14.9	ft	Table 3-3
	$\phi_b BF =$	8.98	kips	$\phi_b BF =$	4.63	kips	Table 3-4
	$\phi_b M_{px} =$	166	kip.ft	$\phi_b M_{px} =$	118	kip.ft	Table 3-5
	Zone =	2		Zone =	2		Table 3-6
	$\phi_b M_{nx} =$	162.2		$M_{nx}/\Omega_b =$	131.6		*Zone 2 Moment*
Check	$\phi_b M_{nx} < \phi_b M_{px}$?	OK		$M_{nx}/\Omega_b < M_{px}/\Omega_b$?	N.G		*Plastic Check*
	$\phi_b M_{nx} > M_u$?	OK		$M_{nx}/\Omega_b > Ma$?	OK		*Check*
USE	**LRFD: W16X26**			**ASD: W16X26**			**ANSWER**

9.32 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	$F_y =$	50	ksi
Ultimate Strength	$Fu =$	70	ksi

Reference: AISC 14th
Section *Eq/Fig/Table*

PROBLEM 9.32

F

9-32. A W21 × 93 has been specified for use on your design project. By mistake, a W21 × 73 was shipped to the field. This beam must be erected today. Assuming that ½ in thick plates are obtainable immediately, select cover plates to be welded to the top and bottom flanges to obtain the necessary section capacity. Use F_y = 50 ksi steel for all materials and assume that full bracing is supplied for the compression flange. Use LRFD and ASD method.

and ASD methods.

Required:

a) Select Plates to be welded on member

Method:

i) Determine information on members

ii) Specify trial plate thickness

iii) Determine plate width

Referenc AISC 14th

Section	*Eq/Fig/Table*
E	

Solution:

LRFD				ASD			
1) Demand:				**Demand:**			
Previous Selection:		**W21X93**				**W21X93**	
Plastic Modulus	Z =	221	in^3		Z =	221	in^3
New Section:		**W21X73**				**W21X73**	
Plastic Modulus	Z =	172	in^3		Z =	172	in^3
Depth:	d =	21.2	in		d =	21.2	in
Reinforcement:		**Plates**				**Plates**	
Number of Plates:		2				2	
Enter trial thickness:	t =	1/2	in		t =	1/2	in
Min Width	w_{min} =	4.52	in		w_{min} =	4.52	in
USE	w =	5.00	in		w =	5.00	in

USE	**Plates**	**1/2**	**5.00**	in	**Plates**	**1/2**	**5.00**	in	**ANSWER**

10.09 MATERIAL PROPERTIES:

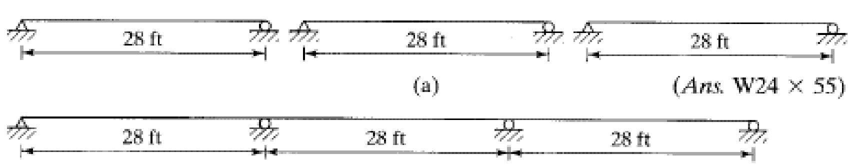
Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi
Ultimate Strength	Fu =	70	ksi

Referenc AISC 14th

Section	*Eq/Fig/Table*
E	

PROBLEM **10.09**

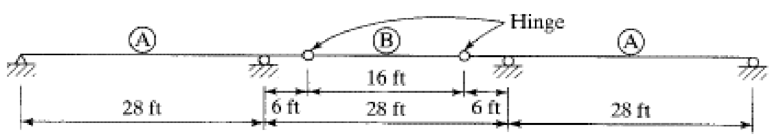
10-9. Three methods of supporting a roof are shown in Fig. P10-9. Using an elastic analysis with factored loads, $F_y = 50$ ksi, and assuming full lateral support in each case, select the lightest section if a dead uniform service load (including the beam self-weight) of 1.5 k/ft and a live uniform service load of 2.0 k/ft is to be supported. Consider moment only.



(a)

(*Ans.* W24 × 55)

(b) (*Ans.* W21 × 44)



(c) (*Ans.* W16 × 26 (B), W21 × 44 (A))

FIGURE P10-9

Required:

a) Select member when using single span, simply supported beams
b) Select member when using continuous span, simply supported beams
c) Select member when using continuous span, with hinges

Method:

i) Determine load demand on member
ii) Determine largest moment
iii) Enter table to obtain member with moment capacity to support demand

Solution:

Reference: AISC 14th

Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED F

Member Length	L =	28	ft	L =	28	ft	
Type of Support:		Simply			Simply		
Dead Load	DL=	1.5	kip/ft	DL=	1.5	kip/ft	
Live Load	LL=	2	kip/ft	LL=	2	kip/ft	
Factors	ϕ_t=	0.9		Ω_t=	1.5		
	ϕ_r=	0.75		Ω_t=	2		

	LRFD			ASD			
1) Demand:				**Demand:**			
Load	Pu =	5	kip/ft	Pa =	3.5	kip/ft	
Moment	Mu =	490	kip.ft	Ma =	343	kip.ft	
1) Capacity:				**Capacity:**			
Beam Selection		**W24X55**			**W21X62**		Table 3-2
Capacity	$\phi_b M_{px}$ =	503	kip.ft	M_{px}/Ω_b =	359	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK		M_{nx}/Ω_b > Ma ?	OK		*Check*
USE	**LRFD:**	**W24X55**		**ASD:**	**W24X55**		**ANSWER**

PART B: CONTINUOUS SPAN, SIMPLY SUPPORTED F

Total Length		84					
Number of Spans		3					
Individual Length	L =	28	ft	40	L =	28	ft
Type of Support:		Case 39				Case 39	

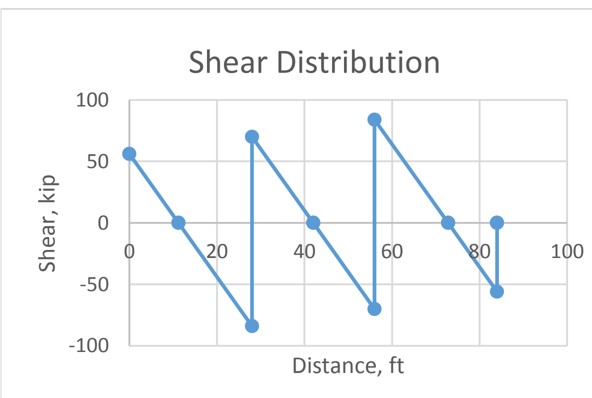
Dead Load	DL=	1.5	kip/ft	DL=	1.5	kip/ft
Live Load	LL=	2	kip/ft	LL=	2	kip/ft
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

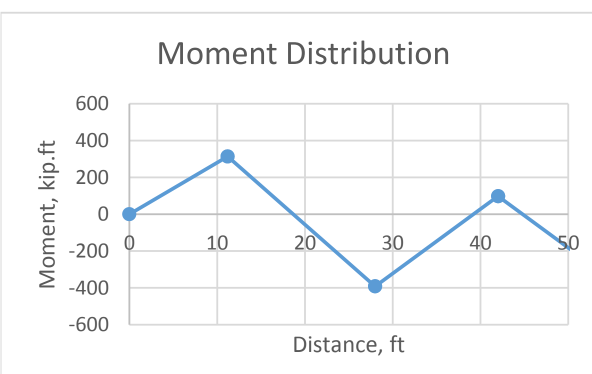
LRFD

1) Demand:

Load Pu = 5 kip/ft

d (ft)	Reaction (kip)	Shear (kip)	Moment (kip.ft)
0	56	56	0
11.2		0	313.6
28	154	70	-392
42		0	98
56	154	84	-392
72.8		0	313.6
84			
84	56	0	0





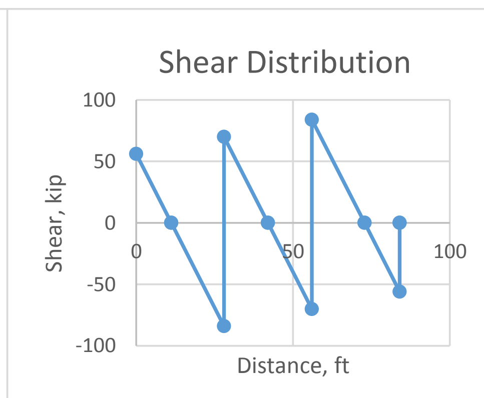
Max (-) Moment	M_{min} =	-392	kip.ft
Max (+) Moment	M_{max} =	313.6	kip.ft
Max absolute	Mu =	392	kip.ft

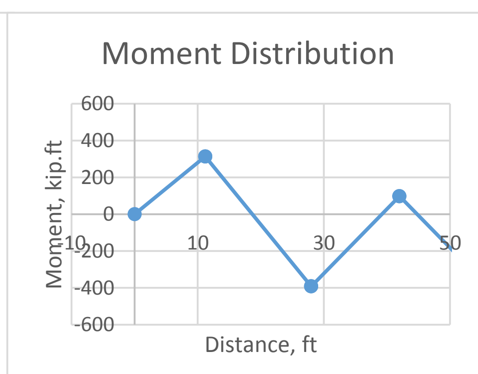
ASD

Demand:

Pa = 3.5 kip/ft

d (ft)	Reaction (kip)	Shear (kip)	Moment (kip.ft)
0	39.2	39.2	0
11.2		0	219.52
28	107.8	49	-274.4
42		0	68.6
56	107.8	58.8	-274.4
72.8		0	219.52
84			
84	39.2	0	0





M_{min} =	-274.4	kip.ft
M_{max} =	219.52	kip.ft
Mu =	274.4	kip.ft

1) Capacity:				**Capacity:**				
Beam Selection		**W21X48**				**W21X55**		Table 3-2
Capacity	$\phi_b M_{px}$ =	14.1	kip.ft		M_{px}/Ω_b =	314	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	N.G			M_{nx}/Ω_b > Ma ?	OK		*Check*
USE	**LRFD:**	**W21X48**			**ASD:**	**W21X55**		**ANSWER**

PART C: CONTINUOUS SPAN, WITH HINGES IN THE MIDDLE

F

Total Length		84	ft			84	ft
Number of Spans		3				3	
Individual Length	L =	28	ft	40	L =	28	ft
Type of Support:		Case 39h				Case 39h	
Distance to hinge		6	ft			6	ft
Distance between hinges		16	ft			16	ft
Dead Load	DL=	1.5	kip/ft		DL=	1.5	kip/ft
Live Load	LL=	2	kip/ft		LL=	2	kip/ft
Factors	ϕ_t=	0.9			Ω_t=	1.5	
	ϕ_r=	0.75			Ω_t=	2	

LRFD | ASD

A - MIDDLE MEMBER:

1) Demand:							
Hinge Member Load	Pu =	80	kip	Pa =	56	kip	
Reaction/Shear:	Vu =	40	kip	Va =	28	kip	
Moment	Mu =	160	kip.ft	Ma =	112	kip.ft	Check Moment

2) Capacity:				**Capacity:**				
Beam Selection		**W16X26**				**W14X30**		Table 3-2
Capacity	$\phi_b M_{px}$ =	166	kip.ft		M_{px}/Ω_b =	118	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK			M_{nx}/Ω_b > Ma ?	OK		*Check*
USE	**LRFD:**	**W16X26**			**ASD:**	**W14X30**		**ANSWER**

B - END MEMBERS:

1) Demand:				**Demand:**			
Load	Pu =	5	kip/ft		Pa =	3.5	kip/ft

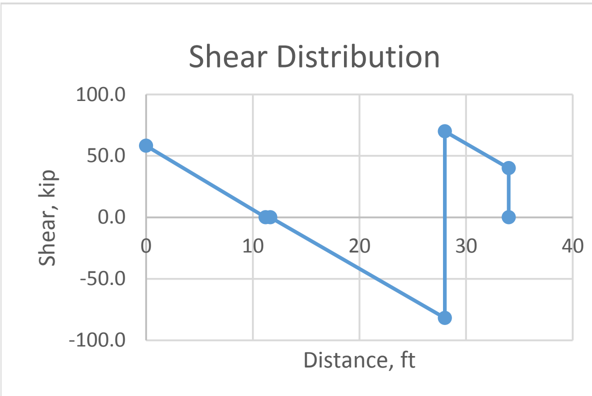
d	Reaction	Shear	Moment

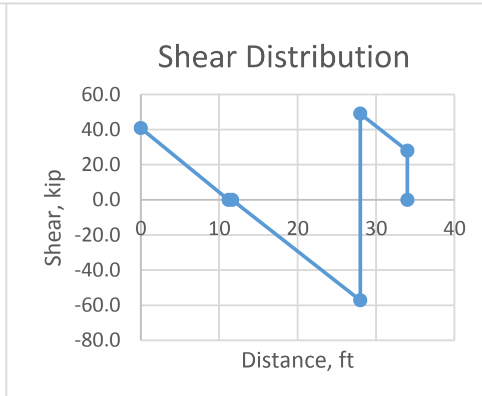
d	Reactior	Shear	Moment

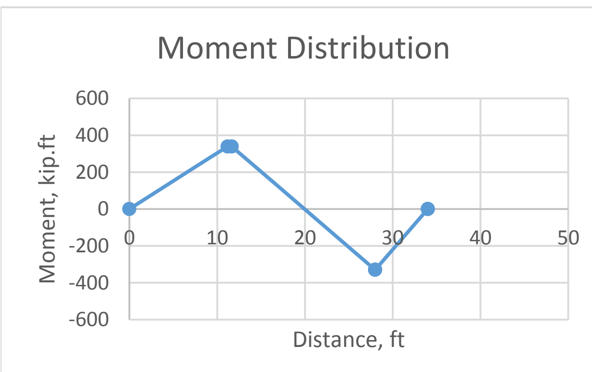
(ft)	*(kip)*	*(kip)*	*(kip.ft)*
0	58.2	58.2	0
11.2		0	338.9
11.64		0	338.9
28	151.8	-81.8	-330
28	151.8	70.0	-330
34		40	0
34		0	0

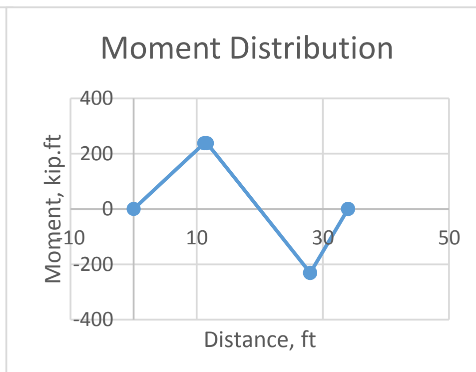
(ft)	*(kip)*	*(kip)*	*(kip.ft)*
0	40.8	40.8	0
11.2		0	237.2
11.64		0	237.2
28	106.3	-57.3	-231
28	106.3	49.0	-231
34		28	0
34		0	0

Distance calculated









Max (-) Moment	M_{min} =	-330	kip.ft	M_{min} =	-231	kip.ft
Max (+) Moment	M_{max} =	338.9	kip.ft	M_{max} =	237.22	kip.ft
Max absolute	Mu =	338.9	kip.ft	Ma =	237.22	kip.ft

1) Capacity: | **Capacity:**

Beam Selection		**W21X44**			**W21X44**		Table 3-2
Capacity	$\phi_b M_{px}$ =	358	kip.ft	M_{px}/Ω_b =	238	kip.ft	Table 3-2
Check	$\phi_b M_{nx} > M_u$?	OK		M_{nx}/Ω_b > Ma ?	OK		*Check*
USE		**LRFD: W21X44**			**ASD: W21X44**		**ANSWER**

10.17 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi
Shear Modulus	G =	11200	ksi
Yield Strength:	F_y =	50	ksi

Reference: AISC 14th

Ultimate Strength	F_u =	70	ksi		Section	Eq/Fig/Table
					F	

PROBLEM 10.17

10-17. A 24-ft, simply supported beam must support a moving concentrated service live load of 50 k in addition to a uniform service dead load of 2.5 k/ft. Using 50 ksi steel, select the lightest section considering moments and shear only. Use LRFD and ASD methods and neglect the beam self-weight. (*Ans.* W24 × 76 LRFD and ASD)

Required:

a) Select lightest section considering moments and shear only

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

Reference: AISC 14th
Section q/Fig/Table
F

PART A: SINGLE SPAN, SIMPLY SUPPORTED

Member Length	L =	24	ft	L =	24	ft
Type of Support:		Simply			Simply	
Dead Load	DL=	2.5	kip/ft	DL=	2.5	kip/ft
Live Load	LL=	50	kip	LL=	50	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD				ASD			
1) Demand:				**Demand:**			
Load	Pu =	80	kip		Pa =	50	kip
Uniform Load	wu =	3	kip/ft		wa =	2.5	kip/ft
Shear	Vu =	116	kip		Va =	80	kip
Moment	Mu =	696	kip.ft		Ma =	480	kip.ft
1) Capacity:				**Capacity:**			
Beam Selection		**W24X76**				**W24X76**	
Capacity	$\phi_b M_{px}$ =	22.4	kip.ft		M_{px}/Ω_b =	76	kip.ft
	$\phi_v V_{nx}$ =	0			V_{nx}/Ω_v =	0	
Check	$\phi_v V_{nx} > V_u$?	OK			V_{nx}/Ω_v > Va ?	OK	
	$\phi_b M_{nx} > M_u$?	N.G			M_{nx}/Ω_b > Ma ?	N.G	

Table 3-2
Table 3-2
Check Shear
Check Flexure

USE **LRFD: W24X76** **ASD: W24X76** **ANSWER**

10.24 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi
Shear Modulus	G =	11200	ksi	G =	11200	ksi
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi

Reference: AISC 14th

Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi

Section | *Eq/Fig/Table*

PROBLEM **10.24** F

10-23. Select the lightest available W sections (F_y = 50 ksi) for the beams and girders shown in Fig. P10-23. The floor slab is 6 in reinforced concrete (weight = 145 lb/ft^3) and supports a 125 psf uniform live load. Assume that continuous lateral bracing of the compression flange is provided. The maximum permissible TL deflection is L/240. (*Ans.* Beam = W21 × 44 LRFD and ASD, Girder = W24 × 62 LRFD and ASD)

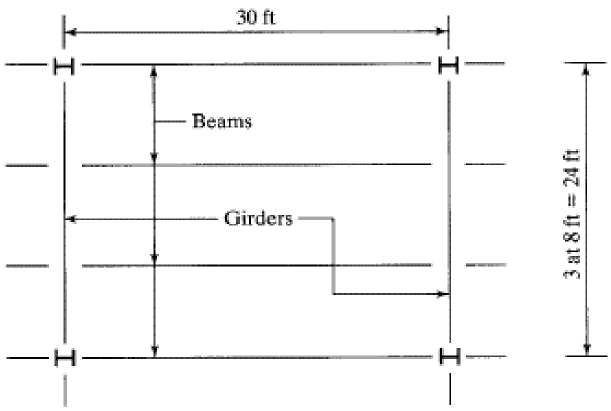


FIGURE P10-23

10-24. Repeat Prob. 10-23 if the live load is 250 psf.

Required:

a) Select lightest section considering moments, shear and TL deflection < L/240

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

PART A: BEAMS E

Beam Length	L =	30	ft	L =	30	ft
Beam Spacing	s =	8	ft	s =	8	ft
Concrete Weight	yc =	145	pcf	yc =	145	pcf
Concrete Slab Thickness		6	in		6	in
Dead Load	DL=	580	kip	DL=	580	kip
Uniform Live Load	Llu=	250	psf			
Live Load	LL=	2000	kip	LL=	2000	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	

LRFD | ASD

1) **Demand:** | **Demand:**

Load	wu =	3896	lb/ft		wa =	2580	lb/ft	
Demand Values:					**Demand Values:**			
Ultimate Moment,	M_u =	**438.3**	kip.ft		Ma =	**290.3**	kip.ft	
Ultimate Shear,	V_u =	**58.4**	kip		Va =	**38.7**	kip	
Allowed Deflection	Δa =	**1.5**	in		Δa =	**1.5**	in	
Beam Selection,	**W:**	**W21X55**				**W21X55**		**Capacity**
	ϕM_n	**473.0**	kip.ft		M_{px}/Ω_b =	**314.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**234.0**	kip		V_{nx}/Ω_v =	**156.0**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**20.8**	in		d	**20.8**	in	
Moment of Inertia	I_x =	**1140.0**	in^4		I_x =	**1140.0**	in^4	
	I_y =	**48.4**	in^4		I_y =	**48.4**	in^4	
Largest M.I =	Ix =	**1140.0**	in^4		Ix =	**1140.0**	in^4	
Beam Deflection:	ΔTL =	1.42	in		ΔTL =	1.42	in	
Design Check:	$\phi M_n > M_u$?	**YES**			M_{nx}/Ω_b > Ma ?	**YES**		**Design Check**
	$\phi V_n > V_u$?	**YES**			V_{nx}/Ω_v > Va ?	**YES**		
	$\Delta_{TL} < \Delta_a$?	**YES**			$\Delta_{TL} < \Delta_a$?	**YES**		
Use	**LRFD:**	**W21X55**			**ASD:**	**W21X55**		**ANSWER**

PART B: GIRDERS E

Girder Length	Lg =	24	ft		Lg =	24	ft	
Tributary Area	At =	192	ft		At =	192	ft	

LRFD | ASD

1) Demand:				**Demand:**				
Uniform Load	wu =	3896	lb/ft		wa =	2580	lb/ft	*From beams above*
Load	Pu =	116.88	lb/ft		Pa =	77.4	lb/ft	
Demand Values:					**Demand Values:**			
Ultimate Moment,	M_u =	**935.0**	kip.ft		Ma =	**619.2**	kip.ft	
Ultimate Shear,	V_u =	**116.9**	kip		Va =	**77.4**	kip	
Allowed Deflection	Δa =	**1.2**	in		Δa =	**1.2**	in	
Beam Selection,	**W:**	**W30X90**				**W30X90**		**Capacity**
	ϕM_n	**1060.0**	kip.ft		M_{px}/Ω_b =	**706.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**374.0**	kip		V_{nx}/Ω_v =	**249.0**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**29.5**	in		d	**29.5**	in	
Moment of Inertia	I_x =	**3610.0**	in^4		I_x =	**3610.0**	in^4	
	I_y =	**115.0**	in^4		I_y =	**115.0**	in^4	

Largest M.I =	Ix =	3610.0	in^4	Ix =	3610.0	in^4	
Beam Deflection:	ΔTL =	0.00	in	ΔTL =	0.61	in	

Design Check:					
Flexure:	$\phi M_n > M_u$?	**YES**	$M_{nx/}\Omega_b$ > Ma ?	**YES**	**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**	$V_{nx/}\Omega_v$ > Va ?	**YES**	
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use | **LRFD: W30X90** | **ASD: W30X90** | **ANSWER**

Check with Self-Weight

Ultimate Moment,	M_u =	**942.8**	kip.ft	Ma =	**627.0**	kip.ft
Ultimate Shear,	V_u =	**118.2**	kip	Va =	**78.7**	kip
Allowed Deflection	Δa =	**0.01**	in	Δa =	**0.62**	in

Design Check:	$\phi M_n > M_u$?	**YES**	$M_{nx/}\Omega_b$ > Ma ?	**YES**	**Design Check**
	$\phi V_n > V_u$?	**YES**	$V_{nx/}\Omega_v$ > Va ?	**YES**	
	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use | **LRFD: W30X90** | **ASD: W30X90** | **ANSWER**

10.27 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th

Section *q/Fig/Table*

PROBLEM **10.27** F

10-27. The beam shown in Fig. P10-27 is a W14 × 34 of A992 steel and has lateral support of the compression flange at the ends and at the points of the concentrated loads. The two concentrated loads are service live loads. Check the beam for shear and for Web Local Yielding and Web Crippling at the concentrated load if l_b = 6 in. Neglect the self-weight of the beam. (*Ans.* Shear and web crippling N.G., web local yielding OK)

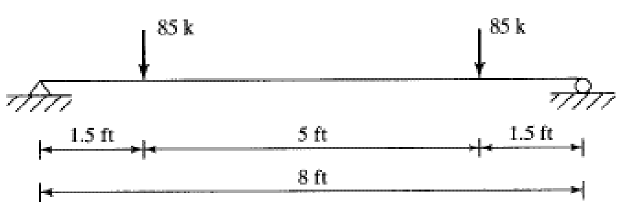


FIGURE P10-27

Required:

a) Select lightest section considering moments, shear
b) Web Local Yielding and Crippling at the concentrated load

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

BEAM SHEAR, MOMENT & DEFLECTION E

Beam Length	L =	8	ft	L =	8	ft
x, Point Load:	Lx =	1.5	ft	Lx =	1.5	
Dead Load	DL =	0	kip	DL =	0	
Live Load	LL=	85	kip	LL=	85	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
Bearing Length:	lb =	6	in	lb =	6	in

LRFD | ASD

1) Demand: | **Demand:**

Load	Pu =	136	kip	Pa =	85	kip	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**204.0**	kip.ft	Ma =	**127.5**	kip.ft	
Ultimate Shear,	V_u =	**136.0**	kip	Va =	**85.0**	kip	
Allowed Deflection	Δa =	**0.4**	in	Δa =	**0.4**	in	
Beam Selection,	**W:**	**W14X34**			**W14X34**		**Capacity**
	ϕM_n	**205.0**	kip.ft	$M_{px/}\Omega_b$ =	**136.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**120.0**	kip	$V_{nx/}\Omega_v$ =	**79.8**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**14.0**	in	d	**14.0**	in	
Moment of Inertia	I_x =	**340.0**	in^4	I_x =	**340.0**	in^4	
	I_y =	**23.3**	in^4	I_y =	**23.3**	in^4	
Largest M.I =	Ix =	**340.0**	in^4	Ix =	**340.0**	in^4	
Beam Deflection:	ΔTL =	0.00	in	ΔTL =	0.00	in	*Case 9* Pg 3-215

Design Check:

Flexure:	$\phi M_n > M_u$?	**YES**	$M_{nx/}\Omega_b$ > Ma ?	**YES**	**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**	$V_{nx/}\Omega_v$ > Va ?	**NO**	**ANSWER**
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use **LRFD: W14X34** | **ASD: W14X34** **ANSWER**

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	**W:**	**W14X34**			**W14X34**		
Beam Depth:	d	**14.0**	in	d	**14.0**	in	
Moment of Inertia	k =	**0.855**	in	k =	**0.855**	in	
Thickness of Web	t_w =	**0.285**	in	t_w =	**0.285**	in	
Thickness of Flange	t_f =	0.455	in	t_f =	0.455	in	
Bearing Length:	N =	6	in	N =	6	in	
Location of P load		18	in		18	in	
Location x > beam d ?		Yes			Yes		
Web Yield Capacity							
	Rn =	146.42		Rn =	146.42		5*k+N * Fyw * tw
	ϕR_n =	146.42		R_n/Ω =	97.61		
Check	Rn < Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		
Location of P load		18	in		18	in	
Location x > beam d/2 ?		Yes			Yes		
Web Resistance Capacity							
	Rn =	161.88		Rn =	161.88		.8*tw^2
Web Res. Strength	ϕR_n =	121.41		R_n/Ω =	80.94		
Check	Rn < Vu ?	N.G.		Rn < Va ?	N.G.		**ANSWER**

10.28 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *q/Fig/Table*

PROBLEM **10.28** F

10-28. A 7-ft beam with full lateral support for its compression flange is supporting a moving concentrated live load of 58 k. Using 50 ksi steel, select the lightest W section. Assume the moving load can be placed anywhere in the middle 5 ft of the beam span. Choose a member based on moment then check if it is satisfactory for shear, and compute the minimum length of bearing required at the supports from the standpoint of web local yielding and web crippling. Neglect self-weight.

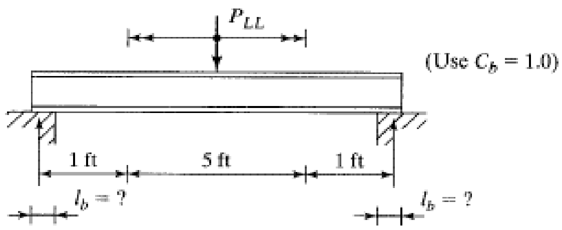


FIGURE P10-28

Required:

a) Select lightest section considering moments, shear
b) Web Local Yielding and Crippling at the concentrated load

Method:

i) Determine load demand on member
ii) Determine largest shear and moment
iii) Enter table to obtain member with moment and shear capacity to support demand

Solution:

BEAM SHEAR, MOMENT & DEFLECTION E

Beam Length	L =	7	ft	L =	7	ft
x, Point Load:	Lx =	3.5	ft	Lx =	3.5	
Dead Load	DL =	0	kip	DL =	0	
Live Load	LL=	58	kip	LL=	58	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
Bearing Length:	Ib =	12	in	Ib =	6	in

LRFD | ASD

1) Demand: | **Demand:**

Load	Pu =	92.8	kip	Pa =	58	kip	
Demand Values:				**Demand Values:**			
Ultimate Moment,	M_u =	**162.4**	kip.ft	Ma =	**101.5**	kip.ft	*Max @ Mid-Span*
Ultimate Shear,	V_u =	**79.5**	kip	Va =	**49.7**	kip	*Max closest to edge*
Allowed Deflection	Δa =	**0.4**	in	Δa =	**0.4**	in	
Beam Selection,	**W:**	**W16X26**			**W16X26**		**Capacity**
	ϕM_n	**166.0**	kip.ft	M_{px}/Ω_b =	**110.0**	kip.ft	*AISC 14th* Table 3-2
	ϕV_n	**106.0**	kip	V_{nx}/Ω_v =	**70.5**	kip	*AISC 14th* Table 3-2
Beam Depth:	d	**15.7**	in	d	**15.7**	in	

		LRFD			ASD		
Moment of Inertia	I_x =	**301.0**	in^4	I_x =	**301.0**	in^4	
	I_y =	**9.6**	in^4	I_y =	**9.6**	in^4	
Largest M.I =	Ix =	**301.0**	in^4	Ix =	**301.0**	in^4	
Beam Deflection:	ΔTL =	0.01	in	ΔTL =	0.01	in	*Case 9* Pg 3-215

Design Check:					
Flexure:	$\phi M_n > M_u$?	**YES**	M_{nx}/Ω_b > Ma ?	**YES**	**Design Check**
Shear:	$\phi V_n > V_u$?	**YES**	V_{nx}/Ω_v > Va ?	**YES**	**ANSWER**
Deflection:	$\Delta_{TL} < \Delta_a$?	**YES**	$\Delta_{TL} < \Delta_a$?	**YES**	

Use | **LRFD: W16X26** | **ASD: W16X26** | **ANSWER**

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING

E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	**W:**	**W16X26**			**W16X26**		
Beam Depth:	d	**15.7**	in	d	**15.7**	in	
Moment of Inertia	k =	**0.747**	in	k =	**0.747**	in	
Thickness of Web	t_w =	**0.250**	in	t_w =	**0.250**	in	
Thickness of Flange	t_f =	0.345	in	t_f =	0.345	in	
Bearing Length:	N =	2.65	in	N =	2.24	in	**ANSWER**
Location of P load		42	in		42	in	
Location x > beam d ?		Yes			Yes		
Web Yield Capacity							
	Rn =	79.81	kip	Rn =	74.69	kip	*Goal Seek*
	ϕR_n =	79.81	kip	R_n/Ω =	49.79	kip	
Check	Rn > Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		
Location of P load		42	in		42	in	
Location x > beam d/2 ?		Yes			Yes		
Bearing Length:	N =	4.25	in	N =	3.45	in	**ANSWER**

Web Resistance Capacity

	Rn =	106.16	kip	Rn =	99.49	kip	.8*tw^2
Web Res. Strength	ϕR_n =	79.62	kip	R_n/Ω =	49.75	kip	*Goal Seek*
Check	Rn < Vu ?	OK		Rn < Va ?	OK		**ANSWER**

USE	W16X26	Nmin =	4.25	in	W16X26 Nmin =	3.5	in	**ANSWER**

10.30 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *Eq/Fig/Table*

PROBLEM 10.30 F

10-30. A W21 × 68 member is used as a simply supported beam with a span length of 12 ft. Determine C_b, since the lateral support of the compression flange is provided only at the ends. The member is uniformly loaded. The loads will produce factored moments of M_{Dx} = 75 ft-k, M_{Lx} = 90 ft-k and M_{Dy} = 15 ft-k, M_{Ly} = 18 ft-k. Is this member satisfactory for bending strength based on the interaction equation in Chapter H of the AISC Specification?

Required:

a) Determine Cb for the member

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Reference: AISC 14th
Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED F

Member Length	L =	12	ft	L =	12	ft
Type of Support:		Simply			Simply	
Dead Moment, x	DLx=	75	kip/ft	DLx=	75	kip/ft
Live Moment, x	LLx=	90	kip/ft	LLx=	90	kip/ft
Dead Moment, y	DLy=	15	kip/ft	DLy=	15	kip/ft
Live Moment, y	LLy=	18	kip/ft	LLy=	18	kip/ft
Factors	ϕ_t=	0.9		Ω_t=	1.669	
	ϕ_r=	0.75		Ω_t=	2	

LRFD — ASD

1) Demand:				**Demand:**		
Moment, x	Mu =	234	kip/ft	Ma =	165	kip/ft
Moment, y	Muy =	46.8	kip/ft	May =	33	kip/ft

1) Capacity: **Capacity:**

Plastic Zones Lengths and Info:							
Beam Selection		**W21X68**			**W21X68**		Table 3-2
Full plastic yield Length	$L_p =$	6.36	ft	$L_p =$	6.36	ft	
LTB Length:	$L_r =$	18.7	ft	$L_r =$	18.7	ft	
	$\phi_b BF =$	18.8	kips	$\phi_b BF =$	18.8	kips	
Capacity	$\phi_b M_{px} =$	600	kip.ft	$M_{px}/\Omega_b =$	399	kip.ft	Table 3-2
	Fy.Zy =	101.67	kip.ft	Fy.Zy =	101.67	kip.ft	
	1.6Fy.Sy =	104.67	kip.ft	1.6Fy.Sy =	104.67	kip.ft	
	$M_{cy} =$	91.5	kip.ft	$M_{cy} =$	60.9	kip.ft	
Check	$\phi_b M_{nx} > M_u$?	**OK**		$M_{nx}/\Omega_b > Ma$?	**OK**		*Check*
Determine Cb:							
Uniform Load	Cb =	1.14		Cb =	1.14		Table 3-1
	Zone =	2		Zone =	2		
	$\phi M_{nx} =$	563.12		$\phi M_{nx} =$	374.49		*BF Equation*
Check	$\phi_b M_{nx} <= Mpx$?	**OK**		$M_{nx}/\Omega_b < Mpx$?	**OK**		*Check*
Equation H1-1b	Ratio =	0.93		Ratio =	0.98		H Eq H1-1b
Check	Eq H1-1b < 1	OK		Eq H1-1b < 1	OK		*Check*
USE	**LRFD: W21X68**			**ASD: W21X68**			**ANSWER**

10.31 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	$F_y =$	50	ksi	$F_y =$	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th

Section *Eq/Fig/Table*

PROBLEM **Capacity** F

10-31. The 30-ft, simply supported beam shown in Fig. P10-31 has full support of its compression flange and is A992 steel. The beam supports a gravity service dead load of 132 lb/ft (includes beam weight) and gravity live load of 165 lb/ft. The loads are assumed to act through the c.g. of the section. Select the lightest available W10 section. (*Ans.* W10 × 22 LRFD, W10 × 26 ASD)

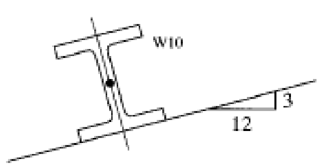


FIGURE P10-31

Required:

a) Determine lightest W10 section

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Reference: AISC 14th
Section *Eq/Fig/Table*

PART A: SINGLE SPAN, SIMPLY SUPPORTED F

Member Length	L =	30	ft	L =	30	ft	
Type of Support:		Simply			Simply		
Dead Load	DL=	132	lb/ft	DLx=	132	lb/ft	
Live Load	LL=	165	lb/ft	LLx=	165	lb/ft	
Factors	ϕ_t=	0.9		Ω_t=	1.669		
	ϕ_r=	0.75		Ω_t=	2		
Datum Rise		3			3		
Datum Run		12			12		
Slope		12.37			12.37		

LRFD | ASD

1) Demand:				**Demand:**			
Load	wu =	422.40	lb/ft	wa =	297.00	lb/ft	
Load in local x	wux =	409.79	lb/ft	wax =	288.13	lb/ft	
Load in local y	wuy =	102.45	lb/ft	way =	72.03	lb/ft	
Moment, x	Mu =	46.10	k.ft	Ma =	32.41	k.ft	
Moment, y	Muy =	11.53	k.ft	May =	8.10	k.ft	
1) Capacity:				**Capacity:**			
Plastic Zones Lengths and Info:							
Beam Selection		**W10X22**			**W10X26**		Table 3-2
Unbraced Length	Lb =	0	ft	Lb =	0	ft	
Full plastic yield Lengtl	L_p =	4.7	ft	L_p =	4.8	ft	Table 3-2
LTB Length:	L_r =	13.8	ft	L_r =	14.9	ft	Table 3-2
	ϕ_bBF =	4.02	kips	ϕ_bBF =	4.34	kips	Table 3-2
Compression Zone	Zone =	1		Zone =	1		
Zone 1 Capacity	ϕM_{nx} =	97.50		ϕM_{nx} =	78.14		Zone 1
Zone 2 Capacity	$\phi_b M_{px}$ =	97.5	kip.ft	M_{px}/Ω_b =	78.1	kip.ft	Zone 2 Table 3-2
Along y axis	Fy.Zy =	25.42	kip.ft	Fy.Zy =	31.25	kip.ft	

	$1.6F_y.S_y$ =	26.47	kip.ft	$1.6F_y.S_y$ =	32.60	kip.ft	
Y axis capacity	M_{cy} =	22.9	kip.ft	M_{cy} =	18.7	kip.ft	*Capacity in y*
Check	$\phi_b M_{nx} > M_u$?	**OK**		M_{ny}/Ω_b > Ma ?	**OK**		*Check*
	$\phi_b M_{nx} <=$ Mpx ?	**OK**		M_{ny}/Ω_b < Mpy ?	**OK**		*Check*
	$\phi_b M_{ny} > M_u$?	**OK**		M_{ny}/Ω_b > Ma ?	**OK**		*Check*
	$\phi_b M_{ny} <=$ Mpy ?	**OK**		M_{ny}/Ω_b < Mpy ?	**OK**		*Check*
Equation H1-1b	Ratio =	0.98		Ratio =	0.85		H Eq H1-1b
Check	Eq H1-1b < 1	OK		Eq H1-1b < 1	OK		*Check*

USE **LRFD: W10X22** **ASD: W10X26** **ANSWER**

10.32 MATERIAL PROPERTIES:

Modulus of Elasticity:	E =	29000	ksi	E =	29000	ksi	
Shear Modulus	G =	11200	ksi	G =	11200	ksi	
Yield Strength:	F_y =	50	ksi	F_y =	50	ksi	
Ultimate Strength	Fu =	70	ksi	Fu =	70	ksi	Reference: AISC 14th
							Section *q/Fig/Table*

PROBLEM **10.32** F

10-32. Design a steel bearing plate from A572 (Grade 50) steel for a W18 × 35 beam, with end reactions of $R_D = 12$ k and $R_L = 16$ k. The beam will bear on a reinforced concrete wall with $f'_c = 3$ ksi. In the direction perpendicular to wall, the bearing plate maximum length of end bearing may not be longer than 6 in. W18 is A992 steel.

Required:

a) Bearing Plate for concrete wall

Method:

i) Determine Member Demand

ii) Determine Member Capacity

Solution:

Beam Length	L =	30	ft	L =	30	ft
Concrete Strength	fc' =	3	ksi	fc' =	3	ksi
Dead Load	DL=	12	kip	DL=	12	kip
Live Load	LL=	16	kip	LL=	16	kip
Factors	ϕ_t=	0.9		Ω_t=	1.5	
	ϕ_r=	0.75		Ω_t=	2	
	ϕ_c=	0.6		Ω_c=	2.5	

LRFD | ASD

1) Demand: | **Demand:**

Load	wu =	40	kip	wa =	28	kip	**LOAD**
Bearing Plate		**Plates**			**Plates**		**BEARING PLATE**
Number of Plates:		1			1		
Area of Plate:	A =	26.14	in^2	A =	27.45	in^2	
Enter trial length:	l =	7	in	l =	7	in	
Min Width	w_{min} =	3.73	in	w_{min} =	3.92	in	
USE	N =	4.00	in	N =	4.00	in	**BEARING LENGTH**
Check Limit	N < 6 in ?	**OK**		N < 6 in ?	**OK**		
Plate Thickness	h =	2.67					
	tmin =	**0.67**	in				
Use Thickness:	t =	**0.75**	in				

USE	**Plates**	**7**	**4.00**	in	**Plates**	**7**	**4.00**	in	**ANSWER**
		thickness	0.75	in		thickness	0.00	0	

BEAM: WEB LOCAL YIELDING & WEB CRIPPLING E

Web Local Yielding

Factor	ϕ=	1		Ω=	1.5		
Web Yield Strength:	Fyw =	50	ksi	Fyw =	50	ksi	
Beam Selection,	**W:**	**W18X35**			**W18X35**		
Beam Depth:	d	**17.7**	in	d	**17.7**	in	
Moment of Inertia	k =	**0.827**	in	k =	**0.827**	in	
Thickness of Web	t_w =	**0.300**	in	t_w =	**0.300**	in	
Thickness of Flange	t_f =	0.425	in	t_f =	0.425	in	
Bearing Length:	N =	4.00	in	N =	4.00	in	
Web Yield Capacity							
	Rn =	91.01	kip	Rn =	91.01	kip	5*k+N * Fyw * tw
	ϕR_n =	91.01	kip	R_n/Ω =	60.68	kip	
Check	Rn < Vu ?	YES		Rn < Va ?	YES		**ANSWER**

Web Crippling

Factor	ϕ_r=	0.75		Ω_t=	2		
Web Resistance Capacity							
	Rn =	72.34	kip	Rn =	72.34	kip	.8*tw^2
Web Res. Strength	ϕR_n =	54.26	kip	R_n/Ω =	36.17	kip	
Check	Rn < Vu ?	OK		Rn < Va ?	OK		**ANSWER**