Flexural Design by using AISC Tables (Table (3.2) (Zx Table) for W-shapes)

W-shape are stored in descending order by strong-axis flexural strength and then grouped in ascending order by weight with the lights W-shape in each rang in bold. Strong-axis available strength in flexure and shear are given for W-shapes with fy =50.
For compact W-shapes, When Lb≤ Lp, the strong-axis available flexural strength $MP/\Omega \; or \; \phi MP$, can be determined using the tabulated strength values. When Lp<Lb≤Lr, linearly interpolate between the available strength at Lp and the available strength at Lr as follows:

Limitation of using table (3.2) (Zx table)

- The W shape is available in table
- Yielding strengthen for steel material must be equal to $(fy = 50)$
- The laterally un supported length (Lb< Lp) (Zone 1) or range between (Lp<Lb≤Lr) (Zone 2).

LRFD	ASD
$\phi Mn = Cb[\phi Mpx - BF(Lb - Lp)] \le \phi Mpx$	$\frac{Mn}{\Omega} = Cb\left[\frac{Mpx}{\Omega} - BF(Lb - Lp)\right] \le \frac{Mpx}{\Omega}$

F_y = 50 ksi

Table 3–2
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF	BF	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W36×800^h	**3650**	**9110**	13700	**5310**	7980	**47.5**	71.4	**14.9**	**94.8**	**64700**	**2030**	**3040**
W36×652^h	**2910**	**7260**	10900	**4300**	6460	**46.8**	70.4	**14.5**	**77.8**	**50600**	**1620**	**2430**
W40×593^h	**2760**	**6890**	10400	**4090**	6140	**55.5**	83.5	**13.4**	**63.8**	**50400**	**1540**	**2310**
W36×529^h	**2330**	**5810**	8740	**3480**	5220	**46.5**	70.0	**14.1**	**64.4**	**39600**	**1280**	**1920**
W40×503^h	**2310**	**5760**	8660	**3460**	5200	**54.7**	82.2	**13.1**	**55.3**	**41600**	**1290**	**1940**
W36×487^h	**2130**	**5310**	7990	**3200**	4800	**46.1**	69.3	**14.0**	**60.0**	**36000**	**1180**	**1770**
W40×431^h	**1960**	**4890**	7350	**2950**	4440	**53.6**	80.6	**12.9**	**49.0**	**34800**	**1110**	**1660**
W36×441^h	1910	4770	7160	2880	4330	45.2	68.0	13.8	55.5	32100	1060	1590
W27×539^h	1890	4720	7090	2740	4120	26.1	39.2	12.9	88.6	25600	1280	1920
W40×397^h	**1800**	**4490**	6750	**2720**	4100	**52.3**	78.7	**12.9**	**46.6**	**32000**	**999**	**1500**
W40×392^h	**1710**	**4270**	6410	**2510**	3780	**60.4**	90.8	**9.33**	**38.3**	**29900**	**1180**	**1760**
W36×395^h	1710	4270	6410	2600	3910	44.7	67.1	13.7	51.0	28500	937	1410
W40×372^h	**1680**	**4190**	6300	**2550**	3830	**51.6**	77.6	**12.7**	**44.5**	**29600**	**943**	**1410**
W14×730^h	1660	4140	6230	2240	3360	7.37	11.1	16.6	275	14300	1380	2060

Example

Use LRFD method to determine the flexural strength of a W14 × 68 of A992 steel material by using table (3.2) if :

a. Continuous lateral support.
b. An unbraced length of 20 ft with Cb = 1.0.
c. An unbraced length of 30 ft with Cb = 1.0.

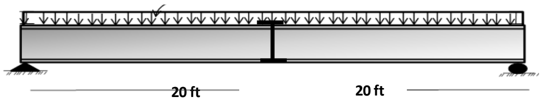
Solution

Steel	fy	fu
A992	50	65

Section	Zx	ϕMpx	Bf	Lp	Lr
W14x68	115	431	7.81	8.69	29.3

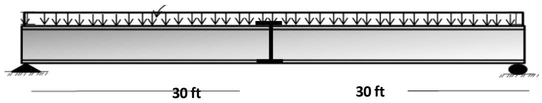
a. Continuous lateral support.
Lb=0
$\phi Mpx = 431\ ft.kip$

b. An unbraced length of 20 ft with Cb = 1.0.



Lp<Lb≤Lr
$\phi Mn = Cb[\phi Mpx - BF(Lb - Lp)] \leq \phi Mpx$
$\phi Mn = 1[431 - 7.81(20 - 8.69)] \leq 431$
=342.66<431
=342.66 ft.kip

c. An unbraced length of 30 ft with Cb = 1.0.



lb>Lr ⟶ Zone 3
30> 29.3
$Mn = FcrxSx < MP$

$$Fcr = \frac{cb\pi^2 E}{(\frac{lb}{rts})^2}\sqrt{(1 + 0.078\frac{Jc}{Sxho}(\frac{lb}{rts})^2)}$$

$$Fcr = \frac{1x\pi^2\, 29000}{(\frac{30x12}{2.8})^2}\sqrt{(1 + 0.078\frac{3.01x1}{103x13.3}(\frac{30x12}{2.8})^2)}$$

= 33.9 ksi

$Mn = 33.9x103 < 5750$
= 3492 <5750

$=\frac{3492}{12}$ = 291 ft.kip

ϕMn=0.9x291=261.9 ft. kip

Example

Use ASD method to determine the flexural strength of a W14 × 74 of A992 steel material if
a. Continuous lateral support.
b. An unbraced length of 15 ft.
c. An unbraced length of 35 ft.

Solution

Steel	fy	fu
A992	50	65

Section	Zx	$Mpx/\Omega b$	Bf	Lp	Lr
W14x74	126	314	5.34	8.76	31

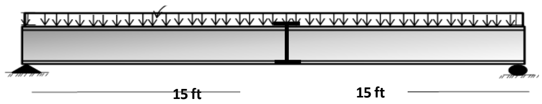
a. Continuous lateral support

L_b = 0 the beams has full laterally supported

$L_b < L_p \longrightarrow$ Zone 1

$$\frac{Mn}{\Omega b} = \frac{Mpx}{\Omega b} = 314\, ft.kip$$

b. An unbraced length of 15 ft.



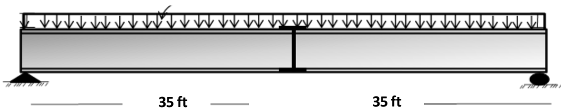
L_b=15 ft
Cb=1.3 table 3.1
Lp<Lb≤Lr

$$\frac{Mn}{\Omega b} = Cb\left[\frac{Mpx}{\Omega b} - BF(Lb - Lp)\right] \le \frac{Mpx}{\Omega b}$$

=1.3x (314-5.34(15-8.76))<314
=364.88>314

Use $\frac{Mn}{\Omega b} = 314$

c. An unbraced length of 35 ft.



b>Lr Zone 3

$$Mn = FcrxSx < MP$$

$$Fcr = \frac{cb\pi^2 E}{(\frac{lb}{rts})^2}\sqrt{(1 + 0.078\frac{Jc}{Sxho}(\frac{lb}{rts})^2)}$$

$$Fcr = \frac{1.3x\pi^2\, 29000}{(\frac{35x12}{2.82})^2}\sqrt{(1 + 0.078\frac{3.87x1}{112x13.4}(\frac{35x12}{2.82})^2)}$$

= 39.2 ksi

$$Mn = 39.2x112 < 6300$$

= 4390.52 <6300

$=\frac{4390.52}{12}$ = 365.87 ft.kip

$$\frac{Mn}{\Omega b} = \frac{Mp}{\Omega b} = \frac{365.87}{1.67} = 219\, ft.kip$$

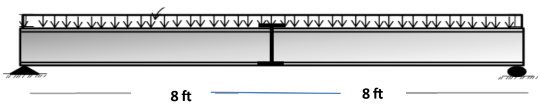
Example

Determine the LRFD design moment capacity of W24x62 with A992 steel material if the laterally unsupported length (Lb=8ft), assume Cb=1

Solution

Steel	fy	fu
A992	50	65

Section	Zx	ϕMpx	Bf	Lp	Lr
W24x62	153	574	24.1	4.87	14.4



Lp<Lb≤Lr   (Zone 2)

$\phi Mn = Cb[\phi Mpx - BF(Lb - Lp)] \leq \phi Mpx$
$\phi Mn = 1[574 - 24.1(8 - 4.87)] \leq 574$
=498.6<574
=498.6 ft.kip

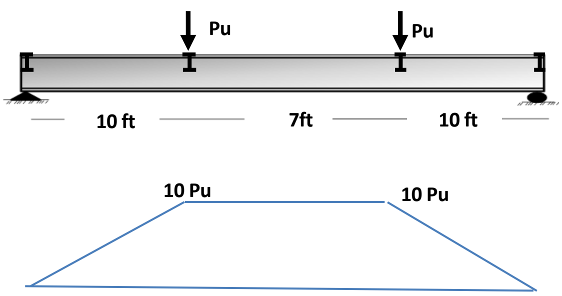
Homework
Determine the ASD allowable moment capacity of W24x62 with A992 steel material if the laterally unsupported length (Lb=8ft), assume Cb=1

Example

Consider the flexural requirements only to determine the allowable point load (Pu), use A992 steel material and W8x58, neglected the self-weight effect. Use LRFD method

Steel	fy	fu
A992	50	65

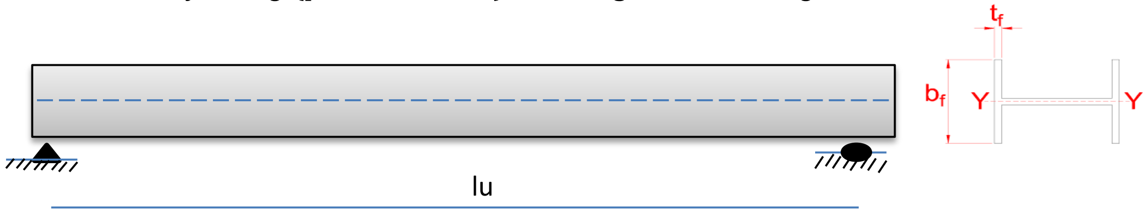
Section	Zx	ϕMpx	Bf	Lp	Lr
W8x 58	59.8	224	2.56	7.42	41.7



Lb_1=7, Lb_2=10
1-portion 1 (Lb=7)
Lb<LP   Zone 1
7<7.42
$\phi Mpx = 224\ ft.kip$
2-portion 2(Lb=8)
Lp<Lb≤Lr   Zone 2
7.42<10≤41.7
Cb=1.67 (table 3.1)
$\phi Mn = Cb[\phi Mpx - BF(Lb - Lp)] \leq \phi Mpx$
$\phi Mn = 1.67[224 - 2.56(10 - 7.42)] \leq 224$
=363>224
Use $\phi Mn = 224$ft.kip
Mu=10Pu
$\phi Mn = Mu$
$224 = 10Pu$
Pu= 22.4 kip

I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

This section applies to I-shaped members and channels bent about their minor axis. The nominal flexural strength, *Mn*, shall be the lower value obtained according to the *limit states* of *yielding* (plastic *moment*) and flange *local buckling*.



1. Yielding

$Mn = Mp = Fy\ Zy \leq 1.6 Fy\ Sy$ (F6-1)

2. Flange Local Buckling

(a) For sections with compact flanges the limit state of yielding shall apply.

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges at *Fy* ≤50ksi.

(b) For sections with non-compact flanges

$Mn = [Mp - (Mp - 0.7 Fy\ Sy) (\frac{\lambda - \lambda p\ f}{\lambda r\ f - \lambda p\ f})]$ (F6-2)

(c) For sections with slender flanges
$Mn = Fcr\ Sy$ (F6-3)
Where:-
$Fcr = \frac{0.69E}{(bf/2tf)^2}$ (F6-4)
$\lambda = b/t$
$\lambda p\ f = \lambda p$, the limiting slenderness for a compact flange, Table B4.1
$\lambda r\ f = \lambda r$, the limiting slenderness for a non-compact flange, Table B4.1
Sy for a channel shall be taken as the minimum section modulus

Flexural Design by using AISC Tables (Table (3.4) (Zy Table) for W-shapes)

W-shape are stored in descending order by weak-axis flexural strength and then grouped in ascending order by weight with the lights W-shape in each rang in bold. Weak-axis available strength in flexure are given for W-shapes with fy =50 ksi(ASTM992).Cb is taken as unity.
For compact W-shapes, When $\lambda \leq \lambda p$, the weak-axis available flexural strength $/\Omega$ or ϕMPy , can be determined using the tabulated strength values. For non -compact Wshape When $\lambda p < \lambda \leq \lambda r$, linearly interpolate between the available strength at λ p and the available strength at λ r as follows:

Limitation of using table (3.4) (Zy table)
- The W shape is available in table
- Yielding strengthen for steel material must be equal to $(fy = 50)$
- For compact section ($\lambda < \lambda$ p)and non-compact section (λ p< $\lambda \leq \lambda$ r)

FLEXURAL DESIGN TABLES 3–23

F_y = 50 ksi

Table 3–4 (continued)
W Shapes
Selection by Z_y

Z_y

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
	in.³	kip-ft ASD	kip-ft LRFD
W14×159	**146**	**364**	**548**
W12×190	143	357	536
W40×278	140	348	523
W30×191	138	344	518
W40×199	137	342	514
W36×256	137	342	514
W24×207	137	342	514
W27×194	136	339	510
W21×201	133	332	499
W14×145	**133**	**332**	**499**
W40×264	132	329	4905
W18×211	132	329	495
W24×192	126	314	473
W12×170	126	314	473
W30×173	123	307	461
W36×232	122	304	458
W27×178	122	304	458
W21×182	119	297	446
W18×192	119	297	446
W40×235	118	294	443
W24×176	115	287	431
W14×132	**113**	**282**	**424**
W12×152	111	277	416
W27×161	109	272	409
W21×166	108	269	405
W36×210	107	267	401
W18×175	106	264	398
W40×211	105	262	394
W24×162	105	262	394
W14×120	**102**	**254**	**383**
W12×136	98.0	245	368
W36×194	97.7	244	366
W27×146	97.7	244	366
W18×158	94.8	237	356
W24×146	93.2	233	350

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
	in.³	kip-ft ASD	kip-ft LRFD
W14×109	**92.7**	**231**	**348**
W21×147	92.6	231	347
W36×182	90.7	226	340
W40×183	88.3	220	331
W18×143	85.4	213	320
W12×120	85.4	213	320
W33×169	84.4	211	317
W36×170	83.8	209	314
W14×99[f]	**83.6**	**207**	**311**
W21×132	82.3	205	309
W24×131	81.5	203	306
W36×160	77.3	193	290
W18×130	76.7	191	288
W40×167	76.0	190	285
W21×122	75.6	189	283
W14×90[f]	**75.6**	**181**	**273**
W12×106	75.1	187	282
W33×152	73.9	184	277
W24×117	71.4	178	268
W36×150	70.9	177	266
W10×112	69.2	173	260
W18×119	69.1	172	259
W21×111	68.2	170	256
W30×148	68.0	170	255
W12×96	67.5	168	253
W33×141	66.9	167	251
W24×104	62.4	156	234
W40×149	62.2	155	233
W21×101	61.7	154	231
W10×100	61.0	152	229
W18×106	60.5	151	227

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
	in.³	kip-ft ASD	kip-ft LRFD
W12×87	**60.4**	**151**	**227**
W36×135	59.7	149	224
W33×130	59.5	148	223
W30×132	58.4	146	219
W27×129	57.6	144	216
W18×97	55.3	138	207
W16×100	54.9	137	206
W12×79	**54.3**	**136**	**204**
W30×124	54.0	135	203
W10×88	53.1	132	199
W33×118	51.3	128	192
W27×114	49.3	123	185
W30×116	49.2	123	185
W12×72	**49.2**	**123**	**185**
W18×86	48.4	121	182
W16×89	48.1	120	180
W10×77	45.9	115	172
W14×82	44.8	112	168
W12×65[f]	**44.1**	**107**	**161**
W30×108	43.9	110	165
W27×102	43.4	108	163
W18×76	42.2	105	158
W24×103	41.5	104	156
W16×77	41.1	103	154
W14×74	40.5	101	152
W10×68	40.1	100	150
W27×94	38.8	96.8	146
W30×99	38.6	96.3	145
W24×94	37.5	93.6	141
W14×68	36.9	92.1	138
W16×67	35.5	88.6	133

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

Example

Using AISC requirements to determine the design moment capacity of W10 × 30 steel beam has 20 ft long and is subjected to bending about its weak axis only, use LRFD method

Steel	fy	fu
A992	50	65

Section	bf/2tf	Sy	Zy
W10x30	5.7	5.75	8.84

$\lambda < 0.38\sqrt{\frac{E}{fy}}$

5.7<915 compact section

$Mn = Mp = Fy\ Zy \le 1.6Fy\ Sy \quad (F\ 6.1)$

$$Mn = \begin{cases} Mp = 50x\ 8.84 = 442/12 = 36.83\ ft.kip \quad \text{control} \\ 1.6x50x5.75 = 460/12 = 38.3\ ft.kip \end{cases}$$

$\phi Mn = \phi Mp = 0.9x36.83 = 33.147\ ft.kip$

or by use table (3.4)

$\phi Mn = 33.2\ ft.kip$

Example

Using AISC requirements to determine the design moment capacity of W12x65[f] steel beam has 20 ft long and is subjected to bending about its weak axis only, use LRFD method

Steel	fy	fu
A992	50	65

Section	bf/2tf	Sy	Zy
W10x30	9.92	29.1	44.1

$\lambda = 9.92$

$\lambda pf = 0.38\sqrt{\frac{E}{fy}}$ λp, the limiting slenderness for a compact flange, Table B4.1

$= 9.15$

$\lambda rf = \sqrt{\frac{E}{fy}}$ λr, the limiting slenderness for a compact flange, Table B4.1

$= 24$

$\lambda pf < \lambda \le \lambda rf$ (non-compact section)

$9.15f < 9.92 \le 24$

$$Mp = \begin{cases} 50x\,44.1 = 2205 \text{ in.kip} & \text{control} \\ 1.6x50x29.1 = 2328 \text{ in.kip} \end{cases}$$

$$Mn = [Mp - (Mp - 0.7Fy\,Sy)\,(\frac{\lambda - \lambda p\,f}{\lambda r\,f - \lambda p\,f})]$$

$$Mn = [2205 - (2205 - 0.7x50x29.1)\,(\frac{9.92 - 9.15}{24 - 9.15})]$$

$Mn = 2143.5 in.kip = 2143.5/12 = 178.625$ ft.kip

$\phi Mn == 0.9x178.625 = 160.763$ ft.kip

By using table (3.4)

$\phi Mn = \phi Mp = 161$ ft.kip

Example

Using AISC requirements to determine the design moment capacity of W12x58 steel beam has 20 ft long and is subjected to bending about its weak axis only, use ASD method

Steel	fy	fu
A992	50	65

Section	bf/2tf	Sy	Zy
W12x58	7.82	21.4	32.5

$\lambda < 0.38\sqrt{\frac{E}{fy}}$

5.7<9.15 compact section

$Mn = Mp = Fy\,Zy \le 1.6Fy\,Sy$ (F 6.1)

$$Mn = \begin{cases} Mp = 50x\,32.5 = 1625/12 = 135.416 \text{ ft.kip} & \text{control} \\ 1.6x50x21.4 = 1712/12 = 142.67 \text{ ft.kip} \end{cases}$$

$\frac{Mn}{\Omega} = \frac{Mp}{\Omega} = \frac{135.416}{1.67} = 81.087$ ft.kip

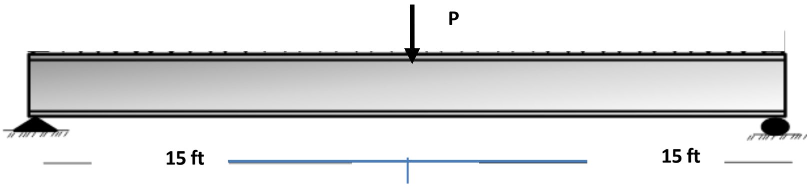
or by use table (3.4)

$\frac{Mn}{\Omega} = 81.1$ ft.kip

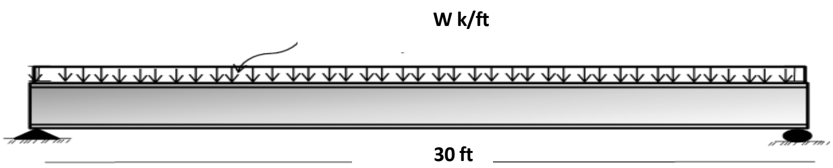
Homework

1-Using AISC requirements to determine the design moment capacity of W18x97 with A992 steel material if the laterally unsupported length (Lb=38ft), assume Cb=1.useLRFD method

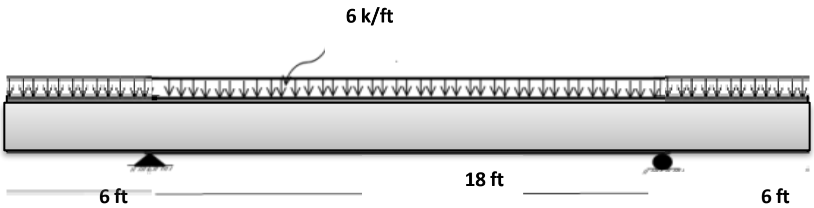
2- Consider the flexural requirements only to determine the allowable point live load (P), use A992 steel material and W10x77, neglected the self-weight effect. Use ASD method



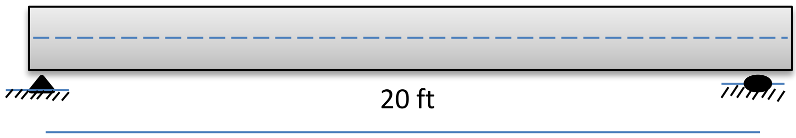
3-W 40x149 has continuous lateral support. If the live load is twice the dead load, what is the maximum total service load (W (k/ft)) that can be supported? Use A992 steel material and LRFD method.



4- W 12x35 has continuous lateral support. The live load is 4k/ft and the dead load is 2 k/ft including self-weight .Use A992 steel material and ASD method is the section adequate?



5-Using AISC requirements to determine the design moment capacity of W12x58 steel beam has 20 ft long, use ASD method



6- Consider the flexural requirements only to determine the allowable point live load (Pu), use A992 steel material and W10x77, The live load is 2k/ft and the dead load is1 k/ft, use LRFD method