Architecture 324

Structures II

Steel Beam Analysis

- Steel Codes: ASD vs. LRFD
- Analysis Methods

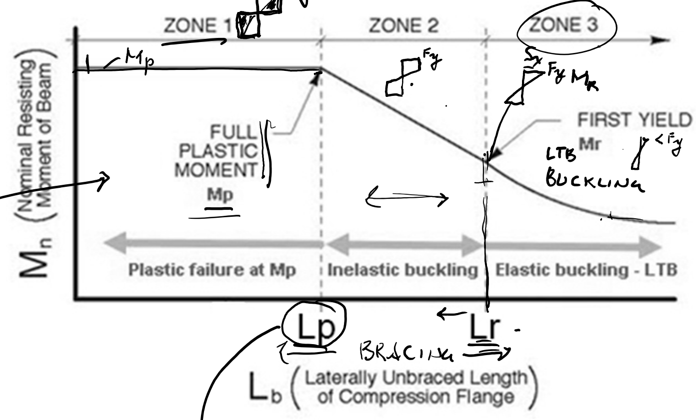
Steel Beams by LRFD

Yield Stress Values

- A36 Carbon Steel $F_y = 36$ ksi
- A992 High Strength $F_y = 50$ ksi

Elastic Analysis for Bending

- **Plastic Behavior (zone 1)**
  $M_n = M_p = F_y Z < 1.5 M_y$
  - Braced against LTB ($L_b < L_p$)
- **Inelastic Buckling "Decreased" (zone 2)**
  $M_n = C_b(M_p-(M_p-M_r)[(L_b-L_p)/(L_r-L_p)]) < M_p$
  - $L_p < L_b < L_r$
- **Elastic Buckling "Decreased Further" (zone 3)**
  $M_{cr} = C_b * \pi/L_b \sqrt{(E*I_y*G*J + (\pi*E/L_b)^2 * I_y C_w)}$
  - $L_b > L_r$

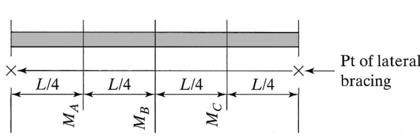


$L_p = 1.76\, r_y \sqrt{E/Fy}$

$M_p = F_y Z_x$

$M_r = 0.7\, F_y S_x$

C_b is LTB modification factor



$$C_b = \frac{12.5\ \text{Mmax}}{2.5\ \text{Mmax} + 3\ \text{MA} + 4\text{MB} + 3\text{MC}}$$

Steel Beams by LRFD

Analysis for Bending

AISC 16th ed.

- Plastic Behavior (zone 1)
  $M_n = M_p = F_y Z < 1.5 M_y$
  - Braced against LTB ($L_b < L_p$)
- Inelastic Buckling "Decreased" (zone 2)
  $M_n = C_b(M_p-(M_p-M_r)[(L_b-L_p)/(L_r-L_p)] < M_p$
  - $L_p < L_b < L_r$
- Elastic Buckling "Decreased Further" (zone 3)
  $M_{cr} = C_b * \pi/L_b \sqrt{(E*I_y*G*J + (\pi*E/L_b)^2 * I_yC_w)}$
  - $Lb > Lr$

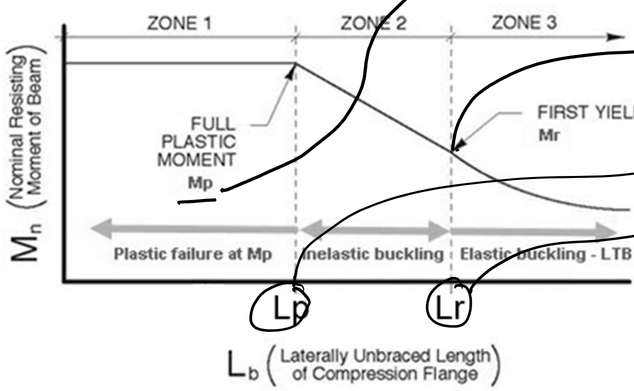


Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Z_x

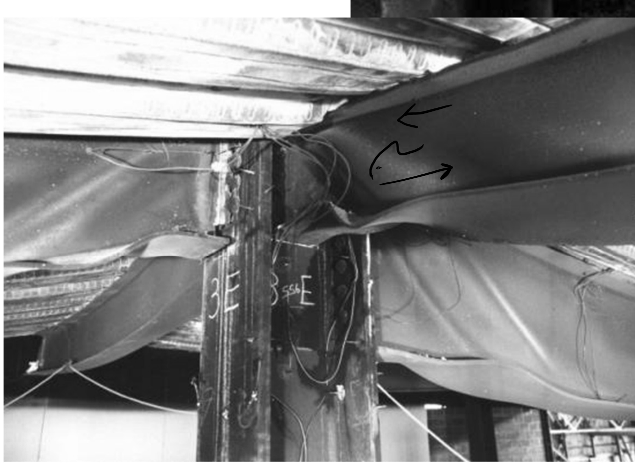
Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
	in.³	kip-ft	kip-ft	kip-ft	kip-ft	kips	kips	ft	ft	in.⁴	kips	kips
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	**420**	**172**	**258**	**9.15**	**13.8**	**5.90**	**17.6**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.1**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Design for Shear

Steel



Design for Shear

Shear stress in steel sections is approximated by averaging the stress in the web:

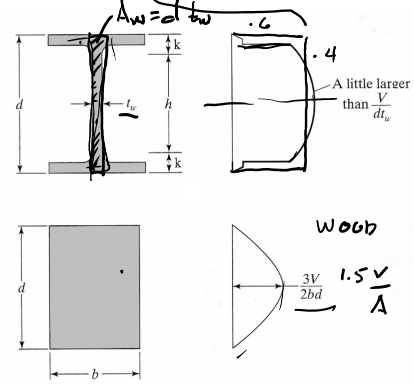
$F_v = V / A_w$

$A_w = d * t_w$

To adjust the stress a reduction factor of 0.6 is applied to F_y

$F_v = 0.6\, F_y$

so, $V_n = 0.6\, F_y\, A_w$ **(Zone 1)**



The equations for the 3 stress zones:
(ϕ in all cases = 1.0)

Zone 1:

WEB YIELDING (Most beam sections fall into this category)

if $\frac{h}{t_w} \le 2.45\sqrt{E/F_y} = 59$ (for 50 ksi steel)

then: $V_n = 0.6\, F_y\, A_w$

Zone 2:

INELASTIC WEB BUCKLING

if $2.45\sqrt{E/F_y} < \frac{h}{t_w} \le 3.07\sqrt{E/F_y} = 74$ (for 50 ksi steel)

then: $V_n = 0.6\, F_y\, A_w\, (2.45\sqrt{E/F}\,) / \frac{h}{t_w}$

Zone 3:

ELASTIC WEB BUCKLING

if $3.07\sqrt{E/F_y} < \frac{h}{t_w} \le 260$

then: $V_n = A_w \left[\frac{4.25\,E}{\left(\frac{h}{t_w}\right)^2}\right]$

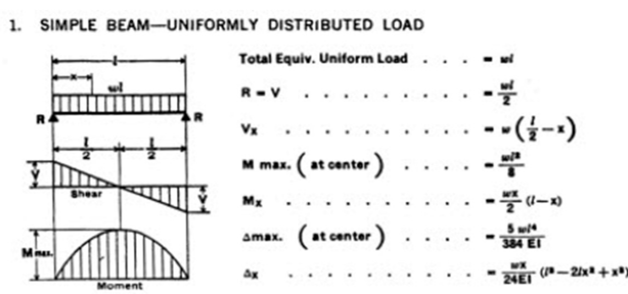
Procedure - Analysis of Steel Beams – **for Zone 1** $L_b < L_p$

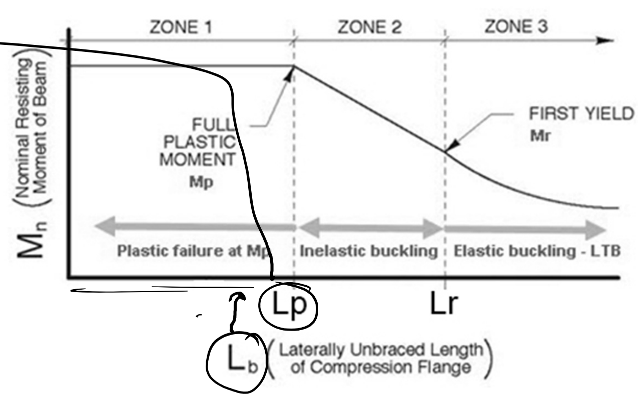
Pass/Fail

Given: yield stress, steel section, loading, bracing (L_b)

Find: pass/fail of section

1. Calculate the factored design load w_u
   $w_u = 1.2w_{DL} + 1.6w_{LL}$
2. Determine the design moment M_u.
   M_u will be the maximum beam moment using the factored loads
3. Insure that $L_b < L_p$ (zone 1)
   $L_p = 1.76\, r_y \sqrt{E/Fy}$
4. Determine the nominal moment, Mn
   $M_n = F_y Z_x$ (look up Z_x for section)
5. Factor the nominal moment
   $øM_n = 0.90\, M_n$
6. Check that $M_u < øM_n$
7. Check shear
8. Check deflection





Example: **Pass/Fail** Analysis of Steel Beams – for Zone 1 $L_b < L_p$

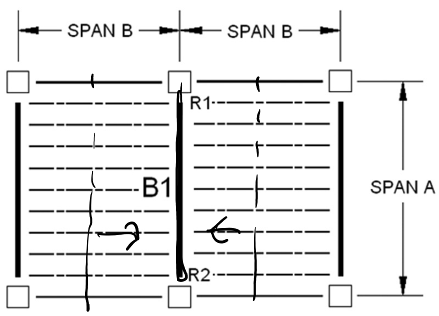
Given: yield stress, steel section, loading, braced 24" o.c.

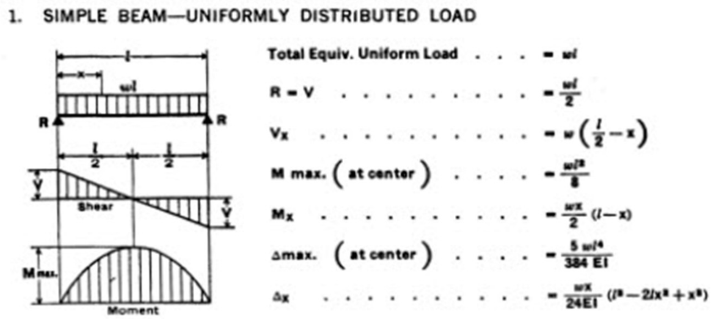
Find: pass/fail of section

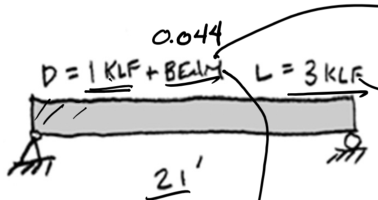
1. Calculate the factored design load w_u

   $w_u = 1.2w_{DL} + 1.6w_{LL}$

2. Determine the design moment M_u. M_u will be the maximum beam moment using the factored loads.







W21×44

A992 STEEL

$F_y = 50$ KSI

FROM TABLE 1-1 AISC $Z_x = 95.4\ in^3$

γ_{DL} γ_{LL}

$w_u = 1.2(1 + .044) + 1.6(3) = 6.05$ KLF

$M_u = \frac{w_u l^2}{8} = \frac{6.05\ KLF \times 21'^2}{8} = 333.5$ K-'

Example: **Pass/Fail** Analysis of Steel Beams – for Zone 1 $L_b < L_p$

3. Insure that $L_b < L_p$ (zone 1) (24")

   $L_p = 1.76\ r_y \sqrt{E/Fy}$

   $L_p = 1.76\ (1.26) \sqrt{29000/50}$

   $L_p = 53.4$ in. > 24 in. ok ✓

   so ZONE 1.

4. Determine the nominal moment, M_n

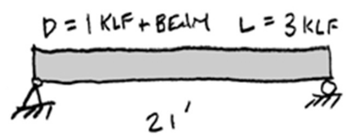
   $M_n = M_p = F_y Z_x$ (for zone 1)

   (look up Z_x for section)

5. Factor the nominal moment

   $øM_n = 0.90\ M_n$

6. Check that $M_u < øM_n$



W21×44

A992 STEEL

$F_y = 50$ KSI

FROM TABLE 1-1 AISC $Z_x = 95.4\ in^3$

$M_n = F_y Z = 50\ ksi \cdot 95.4\ in^3 = 4770$ K-"

$M_n = 4770$ K-" / 12 = 397.5 K-'

$\phi M_n = 0.9\ (397.5) = 357.7$ K-'

DESIGN LOAD M_u — STRENGTH

$M_u = 333.5$ K-' < 357.7 K-' = ϕM_n

∴ PASS ✓

Steel Beams by LRFD

Analysis for Bending

AISC 16th ed.

- Plastic Behavior (zone 1)
  $M_n = M_p = F_y Z$
  - $L_p = 4.45$ ft = 53.4 in. > 24 in. o.c.
  - $\phi_b M_{px} = 358$ k-ft > $M_u = 333.5$ k-ft

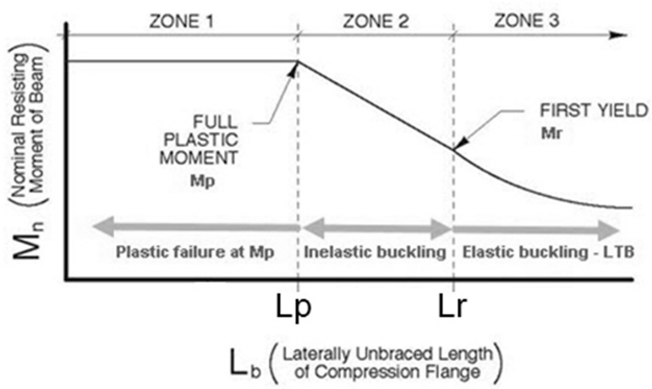


Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi — Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	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
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W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
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W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
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W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
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ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Example: **Pass/Fail** Analysis of Steel Beams – for Zone 1 $L_b < L_p$

7. Check shear for W21x44

CHECK SHEAR:

Zone 1:

WEB YIELDING (Most beam sections fall into this category)

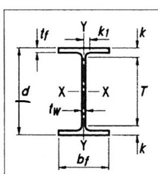
$\frac{20.7}{.35}$ if $\frac{h}{t_w} \le 2.45\sqrt{E/F_y} = 59$ (for 50 ksi steel)

then: $V_n = 0.6 F_y A_w$

FROM AISC TABLE 1-1

$h/t_w = 53.6 < 59$ (zone 1)

Table 1-1 (continued)
W-Shapes
Dimensions



Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83c	24.4	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8	↓	↓
×73c	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8		
×68c	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8		
×62c	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16		
×55c	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16		
×48c,f	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16	↓	↓
W21×57c	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50c	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16	↓	↓
×44c	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W21-W18

Nominal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	Torsional Properties J	Torsional Properties C_w
lb/ft			in.⁴	in.³	in.	in.³	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.⁶
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.5	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.3	0.770	2110

Example: **Pass/Fail** Analysis of Steel Beams – for Zone 1 $L_b < L_p$

7. Check shear (zone 1)

FROM AISC TABLE 1-1

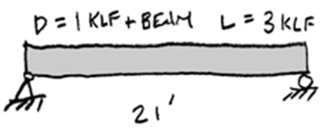
$h/t_w = 53.6 < 59$ (zone 1)

Zone 1:

WEB YIELDING (Most beam sections fall into this category)

if $\frac{h}{t_w} \le 2.45\sqrt{E/F_y} = 59$ (for 50 ksi steel)

then: $V_n = 0.6\, F_y\, A_w$



W21x44

A 992 STEEL

F_y = 50 KSI

FROM TABLE 1-1 AISC $Z_x = 95.4\ in^3$

$w_u = 1.2(1+.044) + 1.6(3) = 6.05$ KLF

CHECK SHEAR: LOAD

$V_u = \frac{w_u \ell}{2} = \frac{6.05(21)}{2} = 63.5^K$

FROM AISC TABLE 1-1

$h/t_w = 53.6 < 59$ (zone 1) ✓

STRENGTH

$V_n = 0.6\, F_y\, A_w = 0.6(50)(20.7 \times 0.35)$ (d, t_w)

$V_n = 217.35^K$

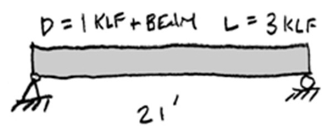
$\phi V_n = 1.0(217.35) = 217.35^K$

$V_u = 63.5^K < 217.3^K = \phi V_n$ ✓

Therefore, pass.

Example: **Pass/Fail** Analysis of Steel Beams – for Zone 1 $L_b < L_p$

8. Check deflection



W21x44

A 992 STEEL

F_y = 50 KSI

FROM TABLE 1-1 AISC $Z_x = 95.4\ in^3$

$w_u = 1.2(1+.044) + 1.6(3) = 6.05$ KLF

$$\Delta_{MAX} = \frac{5 w \ell^4}{384 EI} = \frac{5(3000)\, 21^4\, (1728)}{384(29000000)(843)}$$

(LL; 12^3)

$$= 0.535''$$

$$\frac{\ell}{360} = \frac{21(12)}{360} = 0.7''$$

✓

$$\Delta_{ACTUAL} = 0.535'' < 0.7'' = \Delta_{ALLOWABLE}$$

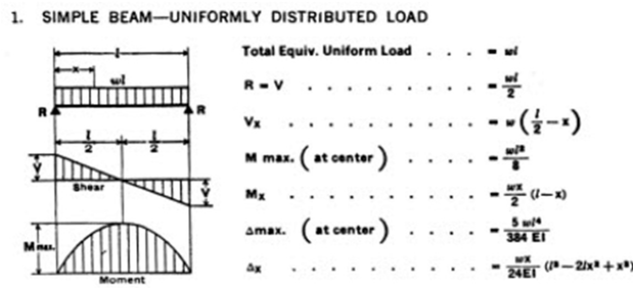
TABLE 1604.3 DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	L	S or W[f]	D + L[d, g]
Roof members:[e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240
Exterior walls:			
With plaster or stucco finishes	—	l/360	—
With other brittle finishes	—	l/240	—
With flexible finishes	—	l/120	—
Interior partitions:[b]			
With plaster or stucco finishes	l/360	—	—
With other brittle finishes	l/240	—	—
With flexible finishes	l/120	—	—
Farm buildings	—	—	l/180
Greenhouses	—	—	l/120

Procedure - Analysis of Steel Beam - **Capacity**

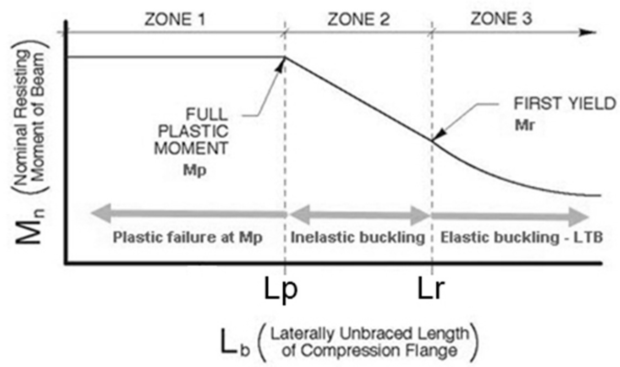
Given: yield stress, steel section, bracing

Find: moment or load capacity



1. Determine the unbraced length of the compression flange (L_b).
2. Find the L_p and L_r values from the AISC Z_x Table 3-2
3. Compare L_b to L_p and L_r and determine which equation for M_n or M_{cr} to be used.
4. Determine the beam load equation for maximum moment in the beam.
5. Calculate load based on maximum moment. $M_u = \phi_b \ M_n$

LOAD STRENGTH



Example – Analysis of Steel Beam - **Capacity**

Given:

Fy = 50 ksi, Fully Braced 20 ft span

Section: W21x44

Find:

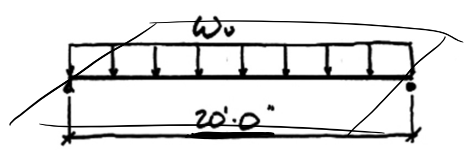
applied live load capacity, w_{LL} in KLF

$w_u = 1.2w_{DL} + 1.6w_{LL}$

w_{DL} = beam + floor = 44plf + 1500plf

1. Find the Plastic Modulus (Z_x) for the given section from the AISC table 1-1
2. Check that $L_b < L_p$ (fully braced – ok)
3. Determine $M_n = M_p = F_y \ Z_x$
4. Set $M_u = \phi_b \ M_n$
   $\phi_b = 0.90$

GIVEN: Fy = 50 ksi
W21x44
FULLY BRACED



FOR A W21x44 FROM TABLE
Zx = 95.4 in³

$M_N = F_Y Z_x = 50 \text{ ksi} \times 95.4 = 4{,}770$ k-in

$M_U = \phi_b \cdot M_N = 0.9 \times 4{,}770$ k-in

$M_U = 4{,}293$ k-in $= 357.75$ k-ft

STRENGTH

Steel Beams by LRFD

Analysis for Bending

AISC 16th ed.

- Plastic Behavior (zone 1)
  $M_n = M_p = F_y Z$
  - L_p = 4.45 ft = 53.4 in.
  - $ø_b M_{px}$ = 358 k-ft = M_u

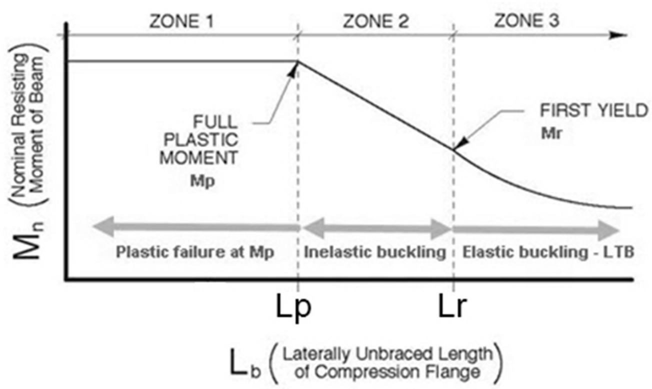


Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi — Z_x

M_p

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	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
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	**420**	**172**	**258**	**9.15**	**13.8**	**5.90**	**17.6**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.1**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112

ASD	LRFD	
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$	[f]Shape exceeds compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Example – Analysis of Steel Beam - **Capacity**

6. Using the maximum moment equation, solve for the factored distributed loading, w_u

$$M_p = M_u = \frac{w_u l^2}{8} \Rightarrow w_u = \frac{8 M_u}{l^2}$$

$$w_u = \frac{8 \times 357.75 \text{ KFT}}{20 \text{ FT}^2}$$

$$w_u = 7.155 \text{ K/FT}$$

7. The applied (unfactored) load
   $w = w_u / (\gamma \text{ factors})$
   $w_u = 1.2wDL + 1.6wLL$

$$w_u = 7.155 \text{ KLF} = 1.2(0.044 + 1.5) + 1.6(w_{LL})$$

$$w_u = 1.853 + 1.6 w_{LL} = 7.155 \text{ KLF}$$

$$w_{LL} = 3.31 \text{ KLF}$$

Steel Beams by LRFD

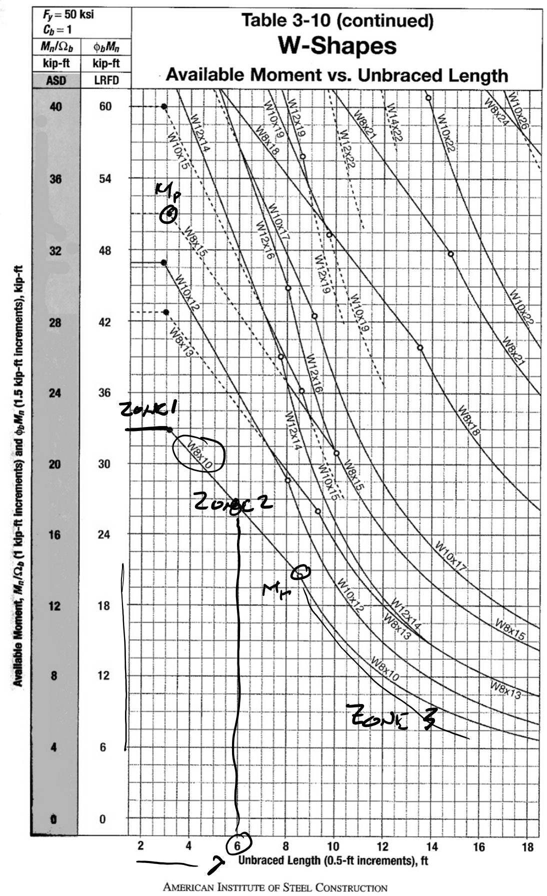
Moment Capacity with L_b Graphs

Analysis for Bending

- Plastic Behavior (zone 1)
  $M_n = M_p$
  Braced against LTB ($L_b < L_p$)

- Inelastic Buckling “Decreased” (zone 2)
  $M_n < M_p$
  $L_p < L_b < L_r$

- Elastic Buckling “Decreased Further” (zone 3)
  $M_n = M_{cr}$
  $L_b > L_r$



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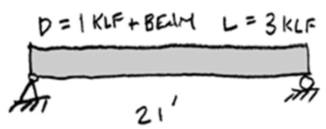
Pass/Fail Analysis of Steel Beams for Zone 1 $L_b < L_p$

Example:

Given: yield stress, steel section, loading, braced @ 24” o.c.

Find: pass/fail of section

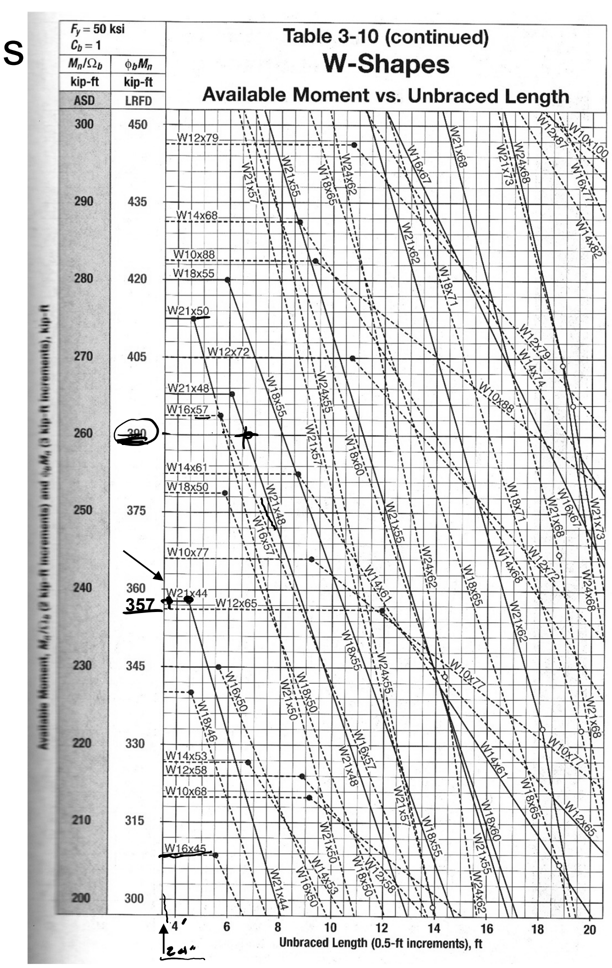


W21x44
A 992 STEEL
F_y = 50 KSI

FROM TABLE 1-1 AISC $Z_x = 95.4 \text{ in}^3$

$w_u = 1.2(1+.044) + 1.6(3) = 6.05 \text{ KLF}$

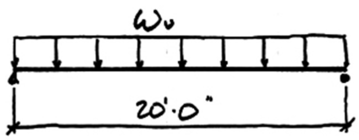
$M_u = \frac{w_u l^2}{8} = \frac{6.05 \text{ KLF} \times 21'^2}{8} = 333.5 \text{ K-'}$



Steel Beams by LRFD

Moment Capacity Graphs

GIVEN: $F_y = 50$ ksi
W21x44
FULLY BRACED

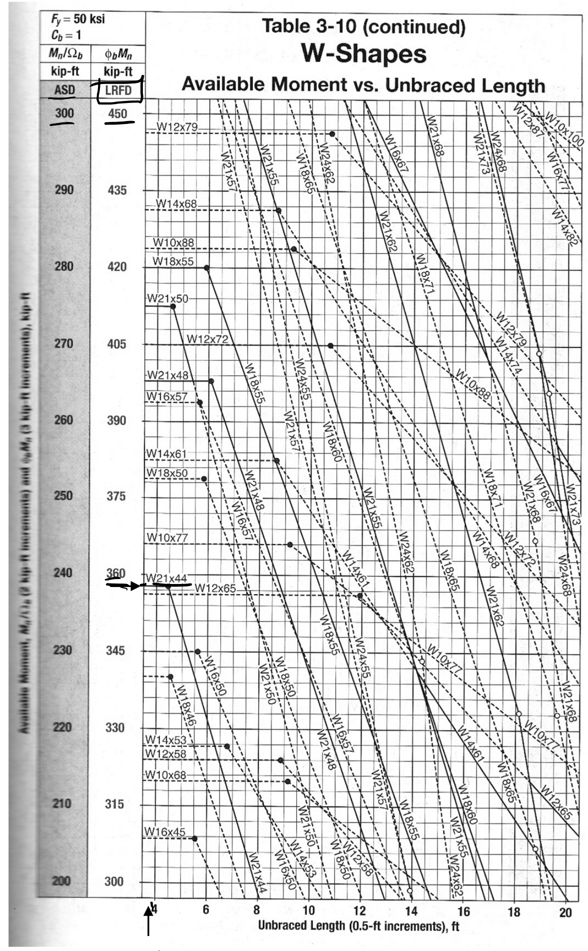


FOR A W21x44 FROM TABLE
$Z_x = 95.4 \text{ in}^3$

$M_n = F_y Z_x = 50 \text{ ksi} \times 95.4 = 4{,}770 \text{ k-in}$

$M_u = \phi_b \cdot M_n = 0.9 \times 4{,}770 \text{ k-in}$

$M_u = 4{,}293 \text{ k-in} = 357.75 \text{ k-ft}$



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