Architecture 324

Structures II

Steel Beam Design

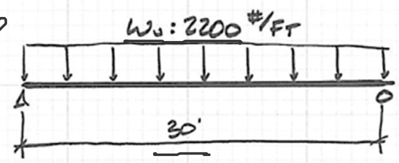
- Design Method
- hot rolled production
- cold formed steel

Design of Steel Beam – Procedure (zone 1)

1. Use the maximum moment equation, and solve for the ultimate moment, Mu.
2. STRENGTH > DESIGN
   Set ϕMn = Mu and solve for Mn
3. Assume Zone 1 to determine Zx required
4. Select the lightest beam with a Zx greater than the Zx required from AISC table
   SIZE
5. Determine if h/tw < 59 CHECK
   (case 1, most common) SHEAR
6. Determine Aw:
   Aw = d tw
7. Calculate Vn:
   Vn = 0.6 Fy Aw
8. Calculate Vu for the given loading
   Vu = w_u L / 2 (e.g. unif. load)
9. Check Vu < ϕ Vn
   ϕ for V = 1.0
10. Check deflection

GIVEN: $F_y = 50$ KSI
FULLY BRACED

w?
?



$$M_u : \frac{w_u l^2}{8} : \frac{2200 \text{ PLF} \cdot 30 \text{ FT}^2}{8}$$

$M_u : 247{,}500$ # $\cdot$ FT $: 247.5$ KFT

$$M_N = M_u / \phi_b : \frac{247.5 \text{ KFT}}{0.90} : 275 \text{ KFT}$$

Design of Steel Beam

Example - Bending

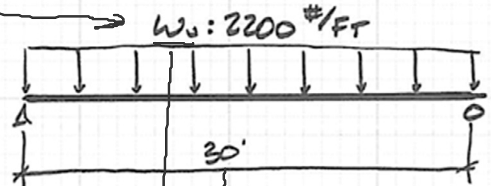
GIVEN: $F_y = 50$ KSI

FULLY BRACED

Applied Load:

DL = 500 plf   LL = 1000 plf

$1.2(500) + 1.6(1000) = 2200$ PLF



1. Use the maximum moment equation, and solve for the ultimate moment, Mu.

$$M_u : \frac{w_u l^2}{8} : \frac{2200 \text{ PLF} \cdot 30 \text{ FT}^2}{8}$$

2. Set ϕMn = Mu and solve for Mn

$$\phi M_n = M_u : 247{,}500 \text{ \#-FT} : 247.5 \text{ KFT}$$

ASSUMING ZONE 1

$$M_N = M_u / \phi_b : \frac{247.5 \text{ KFT}}{0.90} : 275 \text{ KFT}$$

Design of Steel Beam

Example - Bending

3. Determine Zx required (assume zone 1)
   Mn = Fy Zx

4. Select the lightest beam with a Zx greater than the Zx required from AISC table

$$Z_{x\,REQ'D} : \frac{M_N}{F_y} : \frac{275 \text{ KFT} \left(\frac{12''}{\text{FT}}\right)}{50 \text{ KSI}}$$

$$Z_{x\,REQ'D} : 66 \text{ IN}^3$$

SELECT W18x35

Z_x

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

$F_y = 50$ ksi

ϕM_n M_r

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
W21×44	**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
W18×35	**66.5**	**166**	$\boxed{249}$	**101**	151	**8.14**	12.3	$\boxed{4.31}$	**12.3**	**510**	**106**	159
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	~~**54.6**~~	**136**	~~205~~	**84.9**	128	**5.01**	7.55	**5.40**	**15.6**	**340**	**79.8**	120
W16×31	~~**54.0**~~	**135**	~~203~~	**82.4**	124	**6.86**	10.3	**4.13**	**11.8**	**375**	**87.5**	131
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	~~**47.3**~~	**118**	~~177~~	**73.4**	110	**4.63**	6.95	**5.26**	**14.9**	**291**	**74.5**	112
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	~~**44.2**~~	**110**	~~166~~	**67.1**	101	**5.93**	8.98	**3.96**	**11.2**	**301**	**70.5**	106
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9

ASD	LRFD	
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$	[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

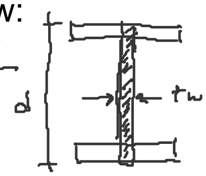
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Design of Steel Beam

Example - Shear

5. Determine if h/tw < 59
   (case 1, most common)

6. Determine Aw:
   Aw = d * tw



FIND h/t_w FROM TABLES FOR A

W18x35

$h/t_w = 53.5 < 59 \; (F_y = 50)$

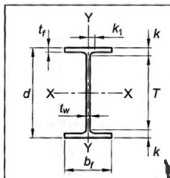


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$t_w/2$	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16	↓	↓
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4	↓	↓
W16×100	29.4	17.0	17	0.585	9/16	5/16	10.4	10 3/8	0.985	1	1.39	1 7/8	1 1/8	13 1/4	5 1/2
×89	26.2	16.8	16 3/4	0.525	1/2	1/4	10.4	10 3/8	0.875	7/8	1.28	1 3/4	1 1/16	↓	↓
×77	22.6	16.5	16 1/2	0.455	7/16	1/4	10.3	10 1/4	0.760	3/4	1.16	1 5/8	1 1/16	↓	↓
×67[c]	19.6	16.3	16 3/8	0.395	3/8	3/16	10.2	10 1/4	0.665	11/16	1.07	1 9/16	1	↓	↓

Table 1-1 (continued)
W-Shapes
Properties

W18–W16

Shape	Compact Section Criteria $b_f/2t_f$	h/t_w	Axis X-X I	S	r	Z	Axis Y-Y I	S	r	Z	r_{ts}	h_o	Torsional Properties J	C_w
			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
W18×46[c]	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
×40[c]	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
×35[c]	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.51	17.3	0.506	1140
W16×100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
×89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
×77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.7	3.57	8590
×67[c]	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.6	2.39	7300

Design of Steel Beam

Example - Shear

5. Determine if h/tw < 59
   (case 1, most common)

6. Determine Aw:
   Aw = d * tw

7. Calculate Vn:
   Vn = 0.6*Fy*Aw

8. Calculate Vu for the given loading
   Vu = w_u L / 2 (unif. load)

9. Check Vu < ϕ_v Vn
   $\phi_v = 1.0$

FIND h/t_w FROM TABLES FOR A

W18x35 ✓

$h/t_w = 53.5 < 59$ ✓

$V_N = 0.6 \cdot F_y \cdot A_w$

$= 0.6 \cdot 50\text{ KSI} \cdot (17.7'' \times .3'')$

STRENGTH $= 159.3$ K

DESIGN LOAD $V_{u_{max}} = \dfrac{\overset{w_u}{2200 \text{ \#/FT}} \cdot \overset{\ell}{30'}}{2} = 33{,}000\text{\#}$

33^K

$V_u \le \phi_v V_N$

$33^K < (1.0)\, 159.3^K = 159.3^K$ OK ✓

Steel Beam Deflection

Deflection limits by application
IBC Table 1604.3

For steel structural members, the DL can be taken as zero (note g)

DL deflection can be compensated for by beam camber

IBC

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 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 and interior partitions:			
With brittle finishes	—	l/240	—
With flexible finishes	—	l/120	—
Farm buildings	—	—	l/180
Greenhouses	—	—	l/120

g–

W 18 × 35

1000 PLF

$$\Delta_{LL} = \frac{5 w_{LL} \ell^4}{384 EI} = \frac{5(1\,\frac{K}{FT})(30\,FT)^4\,1728\,\frac{IN^3}{FT^3}}{384(29000\,\frac{K}{IN^2})(510\,IN^4)}$$

$$= 1.23''$$

$$\frac{\ell}{360} = \frac{30(12)''}{360} = 1.'' < 1.23 \;\therefore\; NG!$$

1000 PLF
500
35

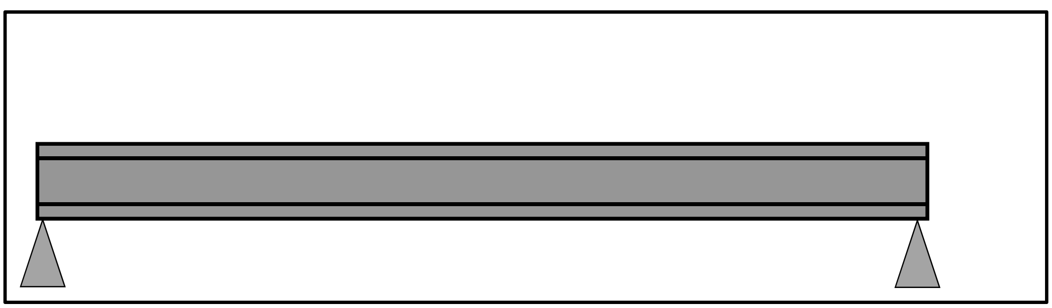
$$\Delta_{DL+LL} = \frac{5(1.535)(30)^4(1728)}{384(29000)(510)} = 1.89''$$

$$\frac{\ell}{240} = \frac{360''}{240} = 1.5 < 1.89 \;\therefore\; NG!$$

TRY W 18×40

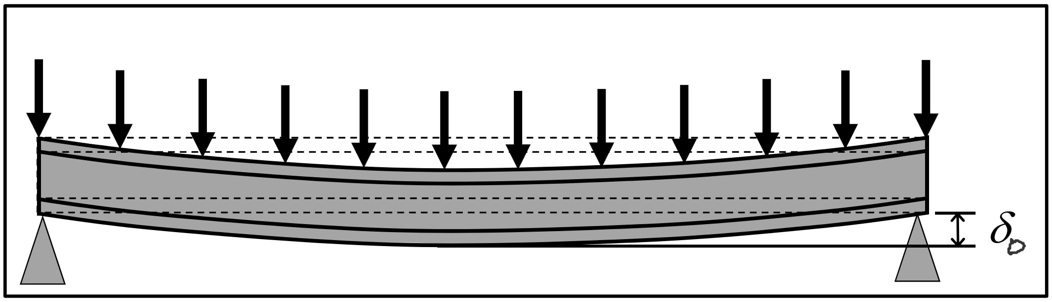
$$\Delta_{LL} = 1.02'' \approx 1.0 = \frac{\ell}{360}$$

$$\Delta_{DL} = 1.54'' \approx 1.5 = \frac{\ell}{240}$$



Beam without Camber

Developed by Scott Civjan
University of Massachusetts, Amherst
For AISC



Results in deflection in floor under Dead Load.
This can affect thickness of slab and fit of non-structural components.

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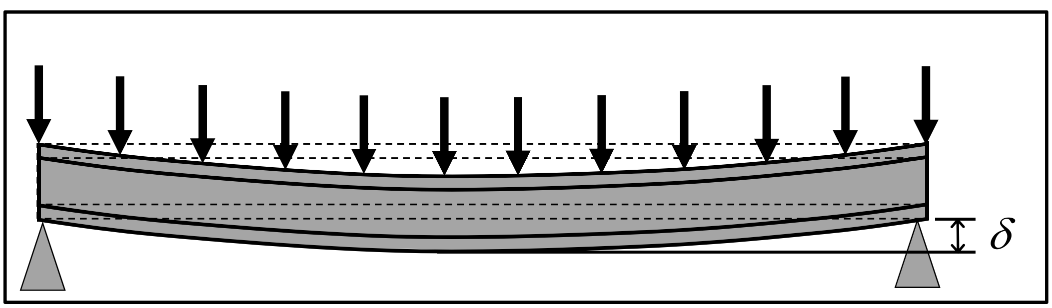
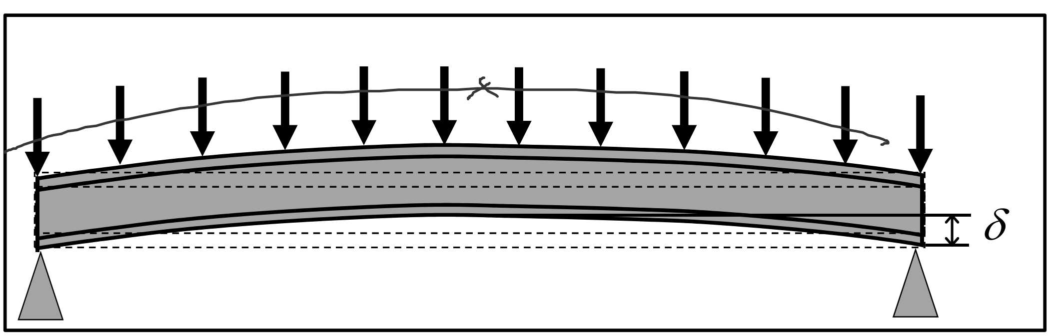


Results in deflection in floor under Dead Load.
This can affect thickness of slab and fit of non-structural components.



Beam with Camber

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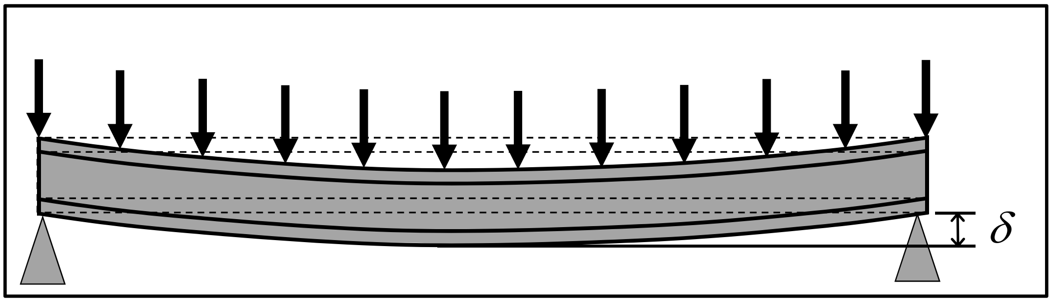
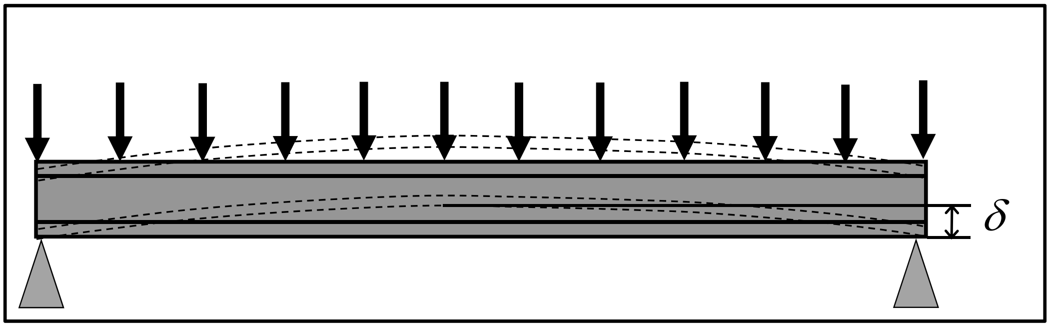
For AISC



Results in deflection in floor under Dead Load.
This can affect thickness of slab and fit of non-structural components.



Cambered beam counteracts service dead load deflection.

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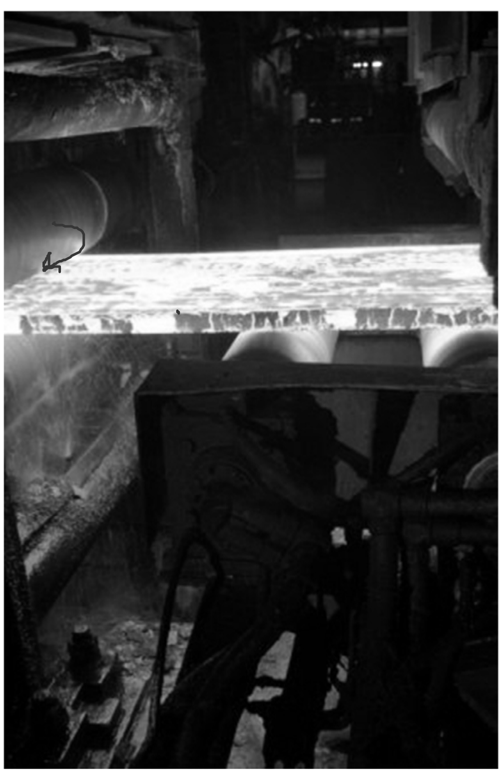
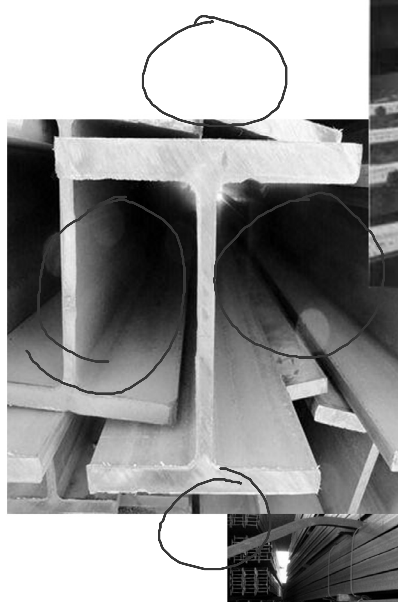
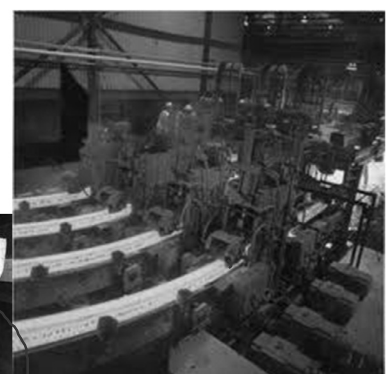
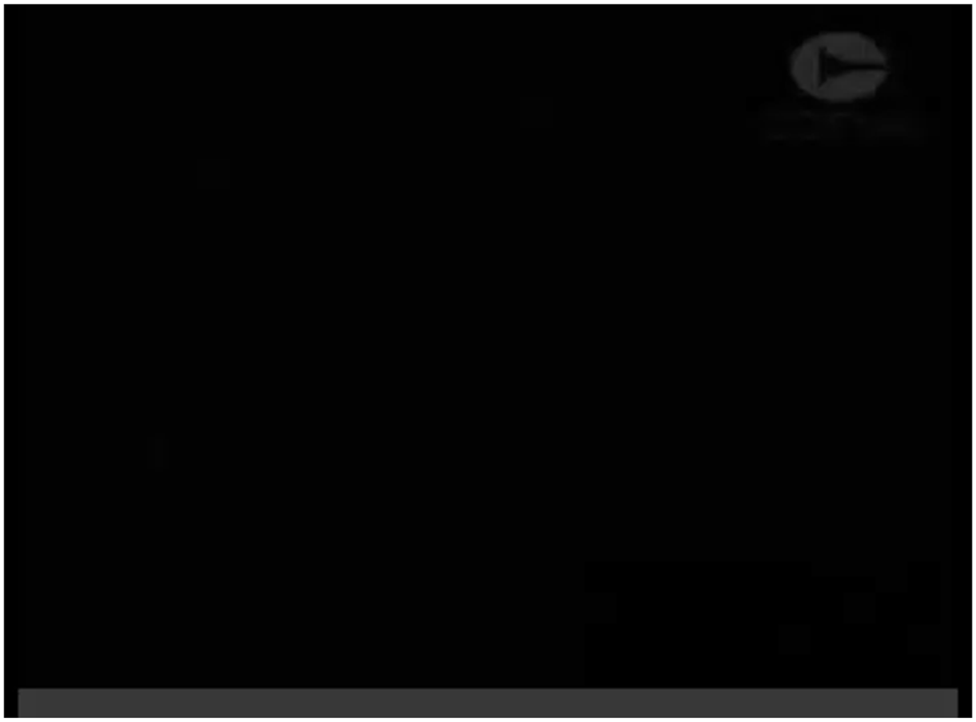
Hot Rolled Shapes







Hot Rolled Shapes



Cold Form Sections

Photos by Albion Sections Ltd, West Bromwich, UK

Cold Form Sections

From:

Building Design Using Cold Formed Steel Sections: Structural Design to BS 5950-5:1998. Section Properties and Load Tables. p. 276

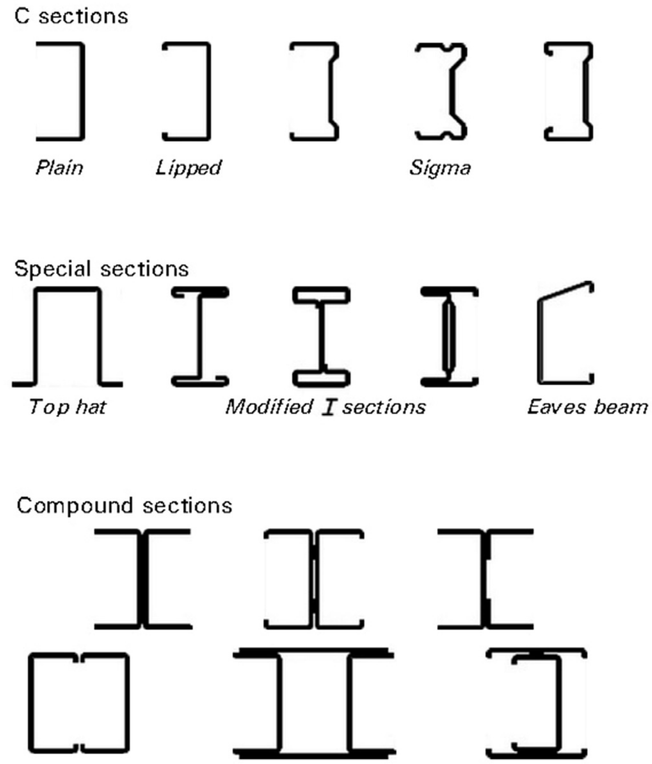
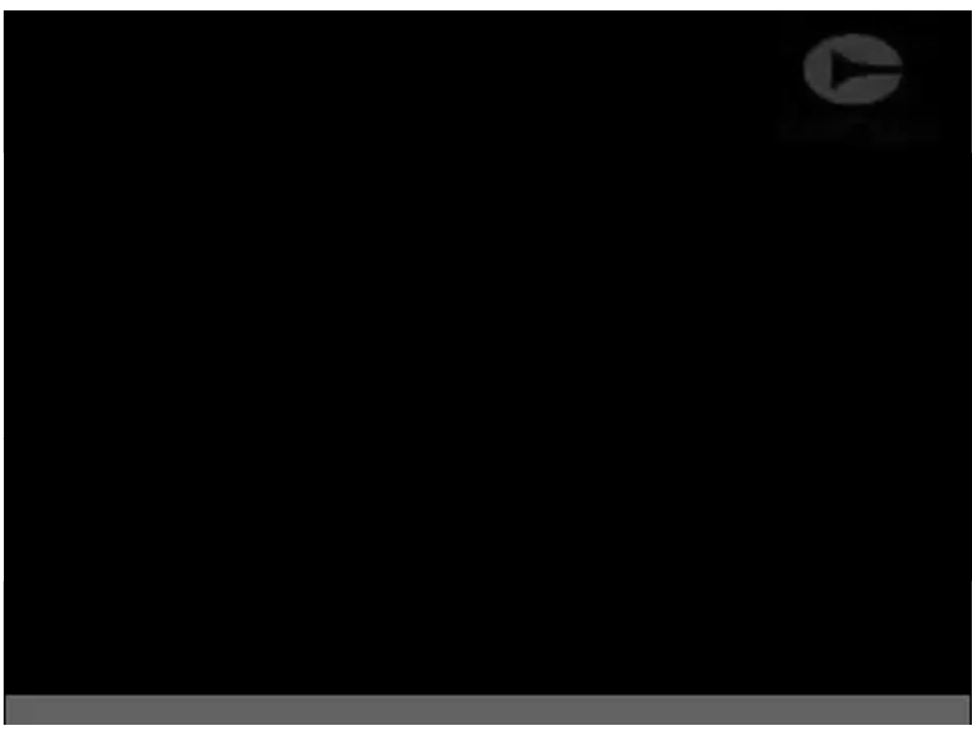
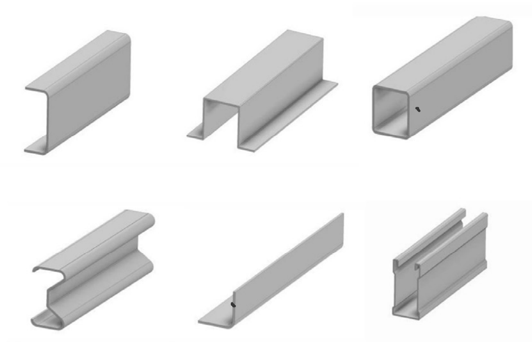


Figure 2.3 *Examples of cold formed steel sections*

Cold Form Sections



Cold Form Sections



STUDS