SUMMARY

OF

AISC-LRFD 2016

ACKNOWLEDGMENTS:

ASA COMPANY WOULD LIKE TO THANKS MR.NADDAFIAN WHO IS AUTHOR OF SUMMARY OF AISC 2005.

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REFERENCES:

ASCE 7–2016
AISC 360–2016
AISC 341–2016
AISC 358–2016
AISC STEEL CONSTRUCTION MANUAL 14th, 2011

1-LOAD COMBINATIONS
(ASCE 7-CHAPTER 2)

IF D,L,E,W,F,T,Lr,S,R,H ARE PRESENT:

1) $1.4(D + F)$
2) $1.2(D + F) + 1.6L + 0.5(Lr \text{ or } S \text{ or } R) + aH$
3) $1.2(D + F) + 1.6(Lr \text{ or } S \text{ or } R) + (bL \text{ or } 0.5W) + aH$
4) $1.2(D + F) + 1W + bL + 0.5(Lr \text{ or } S \text{ or } R) + aH$
5) $0.9D + 1W + aH$
6) $1.2(D + F) + (Ev + (Eh \text{ or } Emh)) + bL + 0.2S + aH$
7) $0.9(D + F) + (-Ev + (Eh \text{ or } Emh)) + aH$

IF ONLY D,L,E,W ARE PRESENT:

1) $1.4D$
2) $1.2D + 1.6L$
3) $1.2D + (bL \text{ or } 0.5W)$
4) $1.2D + bL + 1W$
5) $0.9D + 1W$
6) $1.2D + bL + (Ev + (Eh \text{ or } Emh))$
7) $0.9D + (-Ev + (Eh \text{ or } Emh))$

FOR GENERAL STRUCTURAL INTEGRITY :

1) $1.2D + 1.0N + L + 0.2S$
2) $0.9D + 1N$

- D=DEAD
- L=LIVE
- E_v=VERTICAL SEISMIC FORCE
- E_h=HORIZONTAL SEISMIC FORCE
- E_{mh}=HORIZONTAL SEISMIC FORCE INCLUDING OVER STRENGTH
- F=FLUID
- H=EARTH
- Lr=ROOF LIVE
- R=RAIN
- S=SNOW
- W=WIND
- N=NOTIONAL LOAD

$$a = \begin{cases} 1.6 & \text{For the effect of H adds to the principal load effect.} \\ 0.9 & \text{For the effect of H resists to the principal load effect.} \end{cases}$$

$$b = \begin{cases} 0.5 \text{ if } L < 500\ Kg/m^2 \\ 1.0 \text{ if } L > 500\ Kg/m^2 \end{cases}$$

2-LOCAL BUCKLING *(AISC 360-CHAPTER B)*

bf/2tf LIMIT FOR FLANGE

h/tw LIMIT FOR WEB

- *Compression member*
  - *FLANGE*
    - *rolled* .... $0.56\sqrt{E/Fy} = 16.2$
    - *built-up* .... $\dfrac{kc=4/\sqrt{h/tw}}{0.35<kc<0.76}$ .... $0.64\sqrt{kc.E/Fy}$
  - *WEB (doubly symmetric)* .... $1.49\sqrt{E/Fy} = 43$
  - *SINGLE ANGLE* .... $0.45\sqrt{E/Fy} = 13$

- *Flexural member*
  - *FLANGE*
    - *rolled*
      - *compact* .... $0.38\sqrt{E/Fy} = 11$
      - *non compact* .... $1.0\sqrt{E/Fy} = 28.9$
    - *built-up*
      - *compact* .... $0.38\sqrt{E/Fy} = 11$
      - *non compact* $\dfrac{kc=4/\sqrt{h/tw}}{0.35<kc<0.76}$ $0.95\sqrt{kc.E/F_L}$
  - *WEB*
    - *doubly symmetric*
      - *compact* .... $3.76\sqrt{E/Fy} = 108.5$
      - *non compact* .... $5.70\sqrt{E/Fy} = 164.5$
    - *singly symmetric* .... *refer to table B4.1b*

- *(Seismic) Compression member (I-Shape rolled) (Ductile SMF, SCBF)*
  - *FLANGE* .... $0.32\sqrt{E/Fy\ Ry} = 9.24/\sqrt{Ry}$
  - *WEB*
    - *for* $Ca \leqslant 0.114$ .... $2.57\sqrt{E/Fy\ Ry}(1-1.04Ca) \nearrow 65.4/\sqrt{Ry}$ *for* $Ca=0.114$
    - *for* $Ca>0.114$ $\quad Ca=Pu/0.9Py$ .... $0.88\sqrt{E/Fy\ Ry}(2.68-Ca) \geqslant 1.57\sqrt{E/Fy\ Ry}$
      - $\searrow 42.7/\sqrt{Ry}$ *for* $Ca=1.0$

3-STABILITY OF STRUCTURE (AISC 360-CHAPTER C)

- DIRECT ANALYSIS METHOD:

$$Ni=\begin{cases} 0.002Yi \ \text{(otherwise)} \\ 0.003Yi \ \text{where} \ \dfrac{P_r}{P_{ns}} > 0.5 \ \text{and} \ \tau_b = 1.0 \ \text{was used} \end{cases}$$

$$E.I^* = 0.8E.I$$

$$E.A^* = 0.8E.A$$

- FIRST ORDER DESIGN(APPENDIX 7): (IF Pu ⩽ 0.5Py for all members)

$$Mr = B_1.Mu$$

$$Ni = 2.1(\Delta/L)Yi > 0.0042Yi \ \text{(Ni : horizontal force, Yi : gravity) in all load combo.}$$

(max of all stories)

$$B_1 = \frac{C_m}{1-Pu/Pe}$$

$$P_E = \frac{\pi^2 E.I}{(KL)^2} \qquad K=1$$

$$C_m = \begin{cases} 0.6-0.4(M_1/M_2) \ \cdots\cdots \ \text{no load in span} \\ 1.0 \ \cdots\cdots \ \text{load in span} \end{cases}$$

- AMPLIFIED MOMENTS (APPENDIX 8):

$$M_r = B_1.M_{nt} + B_2.M_{lt}$$

$$P_r = P_{nt} + B_2.P_{lt}$$

B_1 shall be taken as 1 for members not subject to compression.

$Ni = 0.002Yi$ (in gravity only case)

$$B_1 = \frac{C_m}{1-P_r/P_{e1}} \geqslant 1$$

$$P_{e1} = \frac{\pi^2 E.I}{(LC_1)^2}$$

$$B_2 = \frac{1}{1-\dfrac{P_{story}}{P_{e\ story}}} \geqslant 1$$

P_{story} → total vertical load supported by the story

$P_{e\ story}$ → elastic critical buckling strength for the story

$$R_M = 1-0.15\frac{P_{mf}}{P_{story}}$$

P_{mf} → total vertical load in columns in the story that are part of moment frames($R_M = 0$ for braced-frame systems)

$$P_{e\ story} = R_M \frac{HL^{**}}{\Delta_H}$$

Δ_H → first-order interstory drift

$$0.85 \leqslant R_m \leqslant 1$$

1 → no moment frames

0.85 → include moment frames

**H=total story shear

**L=height of story

4-TENSION MEMBERS (AISC 360-CHAPTER D)

Pu (required strength-LRFD) $\leqslant \Phi Pn$ (Design strength)

Pa (required strength-ASD) $\leqslant Pn/\Omega$ (allowable strength)

$$Pn = \min \begin{cases} Fy.Ag & (\Phi=0.90)\ (\Omega=1.67) \quad \text{Yielding on gross section} \\ Fu.Ae & (\Phi=0.75)\ (\Omega=2.00) \quad \text{Rupture on effective net section} \\ \left.\begin{cases} \min \begin{cases} 0.6Fu.Anv+Ubs.Fu.Ant \\ 0.6Fy.Agv+Ubs.Fu.Ant \end{cases} & (\Phi=0.75)\ (\Omega=2.00) \quad \text{if } Ant > 0 \\ \min \begin{cases} Rn=0.6Fy.Agv & (\Phi=1.00)\ (\Omega=1.50) \\ Rn=0.6Fu.Anv & (\Phi=0.75)\ (\Omega=2.00) \end{cases} & \text{if } Ant = 0 \end{cases}\right\} & \text{Block shear strength} \end{cases}$$

$An=Ag-\sum d.t+\sum(s^2/4g).t$, $Ae=U.An$, $d=$ Bolt Diameter + 3 mm

U : shear lag factor (refer to table D3.1)

NOTES:

1) Slenderness ratio not limited. (preferably less than 300)
2) $Ubs=1.00$ for uniform tension, $Ubs=0.5$ for non uniform tension
3) For bolted splice plates $Ae=An\leqslant 0.85Ag$
4) single angle, double angle and WT section shall have $U>0.6$
5) s is center to center distance of sequential hole parallel to force direction.
6) g is center to center distance of sequential hole perpendicular to force direction.

5-1-COMPRESSION MEMBERS (AISC 360–CHAPTER E)

without slender elements

$$Pu \text{ (required strength)} \leqslant \Phi Pn \text{ (Design strength)}$$

$$(\Phi=0.90) \quad (\Omega=1.67)$$

$$Pn=Fcr.Ag \ , \ Fcr=\min\begin{cases} \text{FLEXURAL BUCKLING} \\ \text{FLEXURAL–TORSIONAL BUCKLING}\end{cases}$$

FLEXURAL BUCKLING:

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$$* \ Fcr=\begin{cases}[0.658^{Fy/Fe}]Fy \ \ldots\ldots \text{for } \dfrac{L_c}{r}\leqslant 4.71\sqrt{E/Fy} \\ 0.877Fe \ \ldots\ldots \text{for } \dfrac{L_c}{r}> 4.71\sqrt{E/Fy}\end{cases} \ \ldots\ldots \ \text{for all section types}$$

$$Fe=\frac{\pi^2 E}{\left(\dfrac{L_c}{r}\right)^2}$$

FLEXURAL-TORSIONAL BUCKLING:

for doubly symmetric members use in equation * ...... I ⅂ + ... $Fe=\left[\dfrac{\pi^2.E.Cw}{(L_{cz})^2}+G.J\right]\dfrac{1}{Ix+Iy}$

for singly symmetric members use in equation * ...... $Fe=\left(\dfrac{Fey+Fez}{2H}\right)\left[1-\sqrt{1-\dfrac{4Fey.Fez.H}{(Fey+Fez)^2}}\ \right]$

unsymmetric use in equation * ...... Fe=min root of $(Fe-Fex)(Fe-Fey)(Fe-Fez)-Fe^2.(Fe-Fey)\left(\dfrac{x_\circ}{\bar{r}_\circ}\right)^2-Fe^2.(Fe-Fex)\left(\dfrac{y_\circ}{\bar{r}_\circ}\right)^2=0$

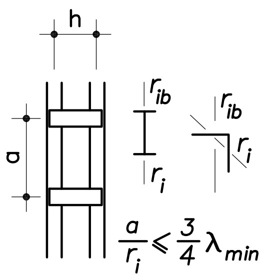
equal legs single angle ...... modified (KL/r) is used in flexural buckling equation as below:

individual member or in planar trusses
$$\begin{cases} KL/r=72+0.75L/r_a \ \ldots\ldots \text{for } L/r_a\leqslant 80 \\ KL/r=32+1.25L/r_a\leqslant 200 \ \ldots\ldots \text{for } L/r_a> 80\end{cases}$$

built–up section ...... modified (KL/r) is used in all above equations as below:

$$\left(\frac{L_c}{r}\right)_m=\begin{cases}\sqrt{\left(\dfrac{L_c}{r}\right)^2+(a/r_i)^2} \ \ldots\ldots \text{intermediate connectors are bolted snug–tight.} \\ \left(\dfrac{L_c}{r}\right) \quad \dfrac{a}{r_i}\leqslant 40 \\ \sqrt{\left(\dfrac{L_c}{r}\right)^2+\left(\dfrac{kia}{r_i}\right)^2} \quad \dfrac{a}{r_i}>40\end{cases}$$

intermediate connectors that are welded or are connected by means of pretensioned bolts with Class A or B faying surfaces.



$$\frac{a}{r_i}\leqslant\frac{3}{4}\lambda_{min}$$

$$Fcrz=\frac{G.J}{Ag.\bar{r}_\circ^2} \ , \ \bar{r}_\circ^2=x_\circ^2+y_\circ^2+\frac{Ix+Iy}{Ag} \ , \ H=1-\frac{x_\circ^2+y_\circ^2}{\bar{r}_\circ^2}$$

$$Fex=\frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2} \ , \ Fey=\frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \ , \ Fez=\left[\frac{\pi^2.E.Cw}{(L_{cz})^2}+G.J\right]\frac{1}{Ag.\bar{r}_\circ^2}$$

$J=\sum b.t^3/3$ (for open section) $\quad Cw\approx Iy.h_\circ^2/4$ (for I shape) $\quad x_\circ, y_\circ \rightarrow$ shear center coordinates

5-2-BUCKLING STRESS (AISC 360-CHAPTER E)

COMPRESSION MEMBERS

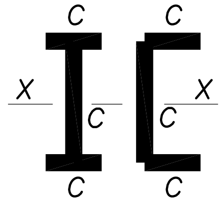
Kl/r	Fcr	Kl/r	Fcr	Kl/r	Fcr	Kl/r	Fcr	Kl/r	Fcr	Kl/r	Fcr	Kl/r	Fcr
1	2400	31	2288	61	1993	91	1587	121	1155	151	774	181	538
2	2400	32	2280	62	1981	92	1573	122	1141	152	764	182	533
3	2399	33	2273	63	1968	93	1558	123	1127	153	754	183	527
4	2398	34	2265	64	1956	94	1544	124	1114	154	744	184	521
5	2397	35	2258	65	1943	95	1529	125	1100	155	734	185	515
6	2396	36	2250	66	1931	96	1515	126	1086	156	725	186	510
7	2394	37	2241	67	1918	97	1500	127	1072	157	716	187	504
8	2392	38	2233	68	1905	98	1486	128	1059	158	707	188	499
9	2390	39	2224	69	1892	99	1471	129	1045	159	698	189	494
10	2388	40	2216	70	1879	100	1457	130	1032	160	689	190	489
11	2386	41	2207	71	1866	101	1442	131	1019	161	681	191	484
12	2383	42	2198	72	1853	102	1427	132	1005	162	672	192	479
13	2380	43	2188	73	1839	103	1413	133	992	163	664	193	474
14	2377	44	2179	74	1826	104	1398	134	979	164	656	194	469
15	2373	45	2169	75	1812	105	1384	135	966	165	648	195	464
16	2370	46	2159	76	1799	106	1369	136	953	166	640	196	459
17	2366	47	2149	77	1785	107	1355	137	940	167	633	197	455
18	2361	48	2139	78	1771	108	1340	138	926	168	625	198	450
19	2357	49	2129	79	1757	109	1326	139	913	169	618	199	445
20	2353	50	2118	80	1743	110	1312	140	900	170	610	200	441
21	2348	51	2108	81	1729	111	1297	141	887	171	603	-	-
22	2343	52	2097	82	1715	112	1283	142	875	172	596	-	-
23	2337	53	2086	83	1701	113	1268	143	863	173	589	-	-
24	2332	54	2075	84	1687	114	1254	144	851	174	583	-	-
25	2326	55	2063	85	1673	115	1240	145	839	175	576	-	-
26	2320	56	2052	86	1659	116	1226	146	828	176	569	-	-
27	2314	57	2041	87	1645	117	1211	147	816	177	563	-	-
28	2308	58	2029	88	1630	118	1197	148	805	178	557	-	-
29	2301	59	2017	89	1616	119	1183	149	795	179	551	-	-
30	2295	60	2005	90	1602	120	1169	150	784	180	544	-	-

6-1-FLEXURAL ELEMENTS (AISC 360-CHAPTER F)

without slender elements

Mu (required strength) $\leqslant \Phi$ *Mn (Design strength)*

$(\Phi=0.90)\quad(\Omega=1.67)$

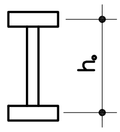


DOUBLY SYMMETRIC (I,[) SHAPE ABOUT MAJOR AXIS
COMPACT FLANGE , COMPACT WEB :

$$Mn=\begin{cases} Mp \quad \text{(Y)} & \text{if} \quad Lb\leqslant Lp \\ C_b\left[Mp-(Mp-0.7Fy.Sx)\left(\dfrac{L_b-L_p}{L_r-L_p}\right)\right]\leqslant Mp \quad \text{(L.T.B)} & \text{if} \quad Lp<Lb\leqslant Lr \\ Fcr.Sx \leqslant Mp \quad \text{(L.T.B)} & \text{if} \quad Lb>Lr \end{cases}$$

$$Fcr=\frac{C_b\pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\,\frac{J.c}{Sx.h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad Mp=Fy.Zx$$

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

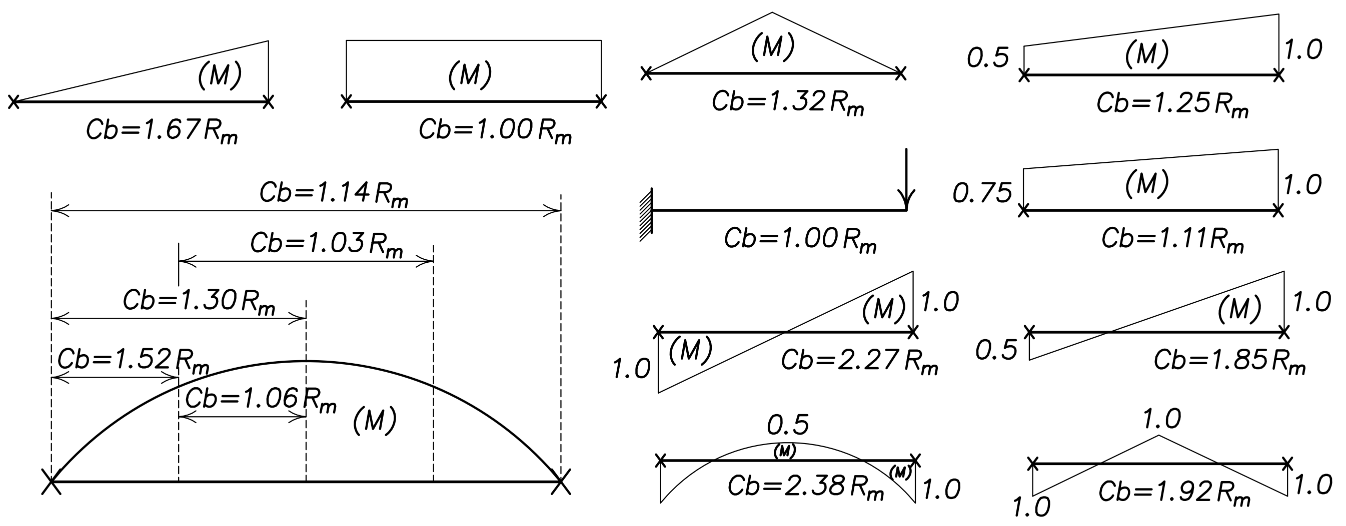


$$L_r=1.95r_{ts}\,\frac{E}{0.7Fy}\sqrt{\frac{J.c}{Sx.h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7Fy}{E}\cdot\frac{Sx.h_o}{J.c}\right)^2}}$$

$$r_{ts}=\sqrt{\frac{\sqrt{Iy.Cw}}{Sx}} \qquad c=\begin{cases} 1.0 \quad \text{I} & \text{for doubly symmetric I shape} \\ \dfrac{h_o}{2}\sqrt{\dfrac{Iy}{Cw}} \quad \text{[} & \text{for channel section} \end{cases}$$

$$C_b=\frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}R_m\leqslant 3.0 \qquad Cw=Iy.h_o^2/4$$

$$R_m=\begin{cases} 1.0 & \text{for doubly symmetric member and singly symmetric single curvature.} \\ 0.5+2\left(\dfrac{I_{yc}}{I_y}\right)^2 & \text{for singly symmetric reverse curvature.} \end{cases}$$



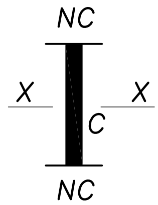
6-2-FLEXURAL ELEMENTS (AISC 360–CHAPTER F)

Mu (required strength) $\leqslant \Phi Mn$ (Design strength)

$(\Phi=0.90)\quad(\Omega=1.67)$

DOUBLY SYMMETRIC (I) SHAPE ABOUT MAJOR AXIS

SLENDER OR NONCOMPACT FLANGE AND COMPACT WEB :



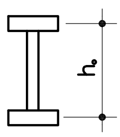
$$Mn=\min\begin{cases} Mp-(Mp-0.7Fy.Sx)\left(\dfrac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right) \text{ .... (NONCOMPACT FLANGE)} \\ \begin{cases} C_b\left[Mp-(Mp-0.7Fy.Sx)\left(\dfrac{L_b-L_p}{L_r-L_p}\right)\right]\leqslant Mp \text{ .... (L.T.B) if } Lp<Lb\leqslant Lr \\ F_{cr}.S_X\leqslant Mp \text{ .... (L.T.B) if } Lb>Lr \end{cases} \\ \dfrac{0.9\ E\ K_C S_X}{\lambda^2} \text{ .... (SLENDER FLANGE)} \end{cases}$$

$\lambda_{pf}=bf/2tf$ limit for compact flange , defined in table B4.16

$\lambda_{rf}=bf/2tf$ limit for noncompact flange , defined in table B4.16

$$F_{cr}=\frac{C_b\pi^2E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\,\frac{J.c}{Sx.h_\circ}\left(\frac{L_b}{r_{ts}}\right)^2}\qquad Mp=Fy.Zx$$

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



$$L_r=1.95r_{ts}\frac{E}{0.7Fy}\sqrt{\frac{J.c}{Sx.h_\circ}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7Fy}{E}\cdot\frac{Sx.h_\circ}{J.c}\right)^2}}$$

$$r_{ts}=\sqrt{\frac{\sqrt{Iy.Cw}}{Sx}}\qquad c=\begin{cases}1.0 \text{ .... I .... for doubly symmetric I shape}\\ \dfrac{h_\circ}{2}\sqrt{\dfrac{Iy}{Cw}} \text{ .... [......... for channel section}\end{cases}$$

$$C_b=\frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}R_m\leqslant 3.0$$

$$R_m=\begin{cases}1.0 \text{ .... for doubly symmetric member and singly symmetric single curvature.}\\ 0.5+2\left(\dfrac{I_{yc}}{I_y}\right)^2 \text{ .... for singly symmetric reverse curvature.}\end{cases}$$

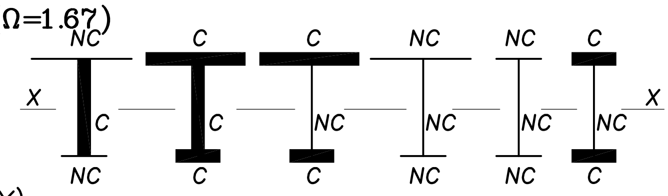
6-3-FLEXURAL ELEMENTS (AISC 360-CHAPTER F)

without slender elements

Mu (required strength) $\leqslant \Phi$ *Mn (Design strength)*

$(\Phi=0.90) \quad (\Omega=1.67)$

OTHER (I) SHAPE ABOUT MAJOR AXIS :



$$M_n=\min\begin{cases} R_{pc}.M_{yc} & \text{(CO.F.Y)} \\ \begin{cases} C_b\left[R_{pc}.M_{yc}-(R_{pc}.M_{yc}-F_L.S_{xc})\left(\dfrac{L_b-L_p}{L_r-L_p}\right)\right]\leqslant R_{pc}.M_{yc} & \text{(L.T.B)} \quad \text{if } L_p<L_b\leqslant L_r \\ F_{cr}.S_{xc}\leqslant R_{pc}.M_{yc} & \text{(L.T.B)} \quad \text{if } L_b>L_r \end{cases} \\ R_{pc}.M_{yc}-(R_{pc}.M_{yc}-F_L.S_{xc})\left(\dfrac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right) & \lambda=b_{fc}/2t_{fc} \text{ for non compact flange} \quad \text{(CO.F.LO.B)} \\ R_{pt}.M_{yt} & \text{(TEN.F.Y)} \quad \text{if } S_{xt}<S_{xc} \end{cases}$$

$$L_p=1.1r_t\sqrt{\frac{E}{F_y}} \quad , \quad M_{yc}=F_y.S_{xc} \quad , \quad M_{yt}=F_y.S_{xt} \quad , \quad M_p=F_y.Z_x$$

$$L_r=1.95r_t\frac{E}{F_L}\sqrt{\frac{J}{S_{xc}.h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_L}{E}\cdot\frac{S_{xc}.h_o}{J}\right)^2}}$$

$$R_{pc}=\begin{cases} \dfrac{M_p}{M_{yc}} \quad M_p\leqslant 1.6S_{xc}.F_y & \text{for } \dfrac{h_c}{t_w}\leqslant\lambda_{pw} \text{ and } \dfrac{I_{yc}}{I_y}>0.23 \\ \dfrac{M_p}{M_{yc}}-\left(\dfrac{M_p}{M_{yc}}-1\right)\left(\dfrac{\lambda-\lambda_{pw}}{\lambda_{rw}-\lambda_{pw}}\right)\leqslant\dfrac{M_p}{M_{yc}} \quad M_p\leqslant 1.6S_{xc}.F_y & \text{for } \dfrac{h_c}{t_w}>\lambda_{pw} \text{ and } \dfrac{I_{yc}}{I_y}>0.23 \\ 1 & \text{for } \dfrac{I_{yc}}{I_y}<0.23 \end{cases}$$

$\lambda = \dfrac{h_c}{t_w}$

$$F_{cr}=\frac{C_b\pi^2E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1+0.078\frac{J}{S_{xc}.h_o}\left(\frac{L_b}{r_t}\right)^2} \qquad * \; J=0 \text{ for } \frac{I_{yc}}{I_y}\leqslant 0.23$$

$$F_L=\begin{cases} 0.7F_y & \text{for } \dfrac{S_{xt}}{S_{xc}}\geqslant 0.7 \\ F_y\dfrac{S_{xt}}{S_{xc}}\geqslant 0.5F_y & \text{for } \dfrac{S_{xt}}{S_{xc}}\leqslant 0.7 \end{cases}$$

$$r_t=\begin{cases} \dfrac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}} & \text{for rectangular flange} \\ \text{radius of gyration of the flange plus one third of web area in compression} & \text{for other flange shapes} \end{cases} \qquad a_w=\frac{h_c.t_w}{b_{fc}.t_{fc}}$$

$$R_{pt}=\begin{cases} \dfrac{M_p}{M_{yt}} & \text{for } \dfrac{h_c}{t_w}\leqslant\lambda_{pw} \text{ and } \dfrac{I_{yc}}{I_y}>0.23 \\ \dfrac{M_p}{M_{yt}}-\left(\dfrac{M_p}{M_{yt}}-1\right)\left(\dfrac{\lambda-\lambda_{pw}}{\lambda_{rw}-\lambda_{pw}}\right)\leqslant\dfrac{M_p}{M_{yt}} & \text{for } \dfrac{h_c}{t_w}>\lambda_{pw} \text{ and } \dfrac{I_{yc}}{I_y}>0.23 \\ 1 & \text{for } \dfrac{I_{yc}}{I_y}<0.23 \end{cases}$$

$\lambda = \dfrac{h_c}{t_w}$

(λ_{rw}, λ_{pw}: see table B4.1b)

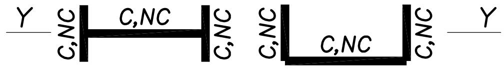
6-4-FLEXURAL ELEMENTS (AISC 360–CHAPTER F)

without slender elements

Mu (required strength) $\leqslant \Phi$ *Mn (Design strength)*

$(\Phi=0.90)$ $(\Omega=1.67)$

(I, [) SHAPE ABOUT MINOR AXIS : ....



$$Mn=\min\begin{cases} Fy.Zy \leqslant 1.6Fy.Sy & \text{(Y)} \quad \text{for compact flange} \\ Mp-(Mp-0.7Fy.Sy)\left(\dfrac{\lambda-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right) & \text{(F.LO.B)} \quad \text{for noncompact flange} \\ F_{cr}.S_y & \text{for slender flange} \end{cases}$$

$\lambda \quad =b_{fc}/2t_{fc}$

λ_{pf} =bf/2tf limit for compact flange , defined in table B4.1b.

λ_{rf} =bf/tf limit for noncompact flange , defined in table B4.1b.

$$F_{cr}=\frac{0.69E}{\left(\dfrac{b_f}{2_{tf}}\right)^2}$$

6-5-IPE PROFILES (AISC 360-CHAPTER F)

FLEXURAL PARAMETERS

IPE	$Z_x(cm^3)$	$Z_y(cm^3)$	Mpx(t.m)	Mpy(t.m)	LP(cm)	Lr(cm)
8	23.2	5.82	0.56	0.14	54	295
10	39.4	9.15	0.95	0.22	64	315
12	60.7	13.6	1.46	0.33	74	328
14	88.3	19.2	2.12	0.46	85	348
16	124	26.1	2.98	0.63	95	375
18	166	34.6	3.98	0.83	105	398
20	221	44.6	5.30	1.07	114	431
22	285	58.1	6.84	1.39	127	462
24	367	73.9	8.81	1.77	138	503
27	484	96.9	11.62	2.33	155	535
30	628	125	15.07	3.00	172	573
33	804	154	19.30	3.70	182	606
36	1019	191	24.46	4.58	194	640
40	1307	229	31.37	5.50	203	664
45	1702	276	40.85	6.62	211	681
50	2194	336	52.66	8.06	220	706
55	2787	400	66.89	9.60	229	733
60	3512	486	84.29	11.66	239	766

6-6-INP PROFILES *(AISC 360-CHAPTER F)*

FLEXURAL PARAMETERS

INP	$Z_x(cm^3)$	$Z_y(cm^3)$	Mpx(t.m)	Mpy(t.m)	LP(cm)	Lr(cm)
80	22.7	5.0	0.54	0.12	47	288
100	39.7	8.1	0.95	0.19	55	311
120	63.5	12.4	1.52	0.30	63	277
140	95.2	17.9	2.28	0.43	72	368
160	136	24.9	3.26	0.60	80	394
180	187	33.2	4.49	0.80	88	424
200	249	43.5	5.98	1.04	96	454
220	323	55.7	7.75	1.34	104	483
240	411	70.0	9.86	1.68	113	518
260	513	85.9	12.31	2.06	119	544
280	631	103	15.14	2.47	126	575
300	761	121	18.26	2.90	131	599
320	913	143	21.91	3.43	137	625
340	1078	166	25.87	3.98	144	654
360	1274	194	30.58	4.66	149	682
380	1480	221	35.52	5.30	155	712
400	1712	253	41.09	6.07	161	734
450	2394	345	57.46	8.28	176	804
500	3235	456	77.64	10.94	191	874
550	4229	592	101.50	14.21	206	920

	SUMMARY OF AISC-LRFD 2016	DATE: 1396/11/24 PAGE: 15/28

6-7-UNP PROFILES (AISC 360-CHAPTER F)

FLEXURAL PARAMETERS

UNP	Zx(cm^3)	Zy(cm^3)	Mpx(t.m)	Mpy(t.m)	LP(cm)	Lr(cm)
80	29.6	12	0.71	0.29	68	572
100	45.2	16.1	1.08	0.39	75	517
120	66.3	21.2	1.59	0.50	81	520
140	93.9	28.2	2.25	0.68	89	517
160	125	35	3.00	0.84	96	516
180	162	42.9	3.89	1.03	103	527
200	205	51.7	4.92	1.24	109	530
220	262	64.2	6.29	1.54	117	555
240	320	75.7	7.68	1.82	123	567
260	396	91.6	9.50	2.20	130	592
280	478	110	11.47	2.64	140	610
300	571	130	13.70	3.12	148	628
320	730	152	17.52	3.65	143	710
350	781	142	18.74	3.41	139	628
380	879	149	21.10	3.58	141	584
400	1081	192	25.94	4.61	155	649

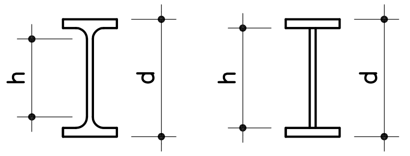
6-8-IPB PROFILES *(AISC 360-CHAPTER F)*

FLEXURAL PARAMETERS

IPB	$Zx(cm^3)$	$Zy(cm^3)$	Mpx(t.m)	Mpy(t.m)	LP(cm)	Lr(cm)
100	104	51.4	2.50	1.23	130	410
120	165	81	3.96	1.94	157	465
140	245	120	5.88	2.88	184	523
160	354	170	8.50	4.08	208	587
180	481	231	11.54	5.54	234	647
200	643	306	15.43	7.34	260	711
220	827	394	19.85	9.46	287	774
240	1053	498	25.27	11.95	312	839
260	1283	602	30.79	14.45	337	895
280	1534	718	36.82	17.23	364	951
300	1869	870	44.86	20.88	389	1017
320	2149	939	51.58	22.54	388	1017
340	2408	986	57.79	23.66	386	1012
360	2683	1032	64.39	24.77	384	1007
400	3232	1104	77.57	26.50	380	994
450	3982	1198	95.57	28.75	376	982
500	4815	1292	115.56	31.00	373	972
550	5591	1341	134.18	32.18	368	958
600	6425	1391	154.20	33.38	363	946
650	7320	1441	175.68	34.58	358	935
700	8327	1495	199.85	35.88	352	924
800	10230	1553	245.52	37.27	343	902
900	12580	1658	301.92	39.79	335	890
1000	14860	1716	356.64	41.18	327	875

7-DESIGN FOR SHEAR *(AISC 360-CHAPTER G)*

Vu (required strength) $\leqslant \Phi$ *Vn (Design strength)*

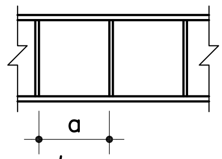


MAJOR AXIS SHEAR STRENGTH OF WEBS WITHOUT TENSION FIELD ACTION :

$$V_n = 0.6F_y.A_w.C_{V1}$$

$$C_{V1} = \begin{cases} \text{for rolled I shape section with } h/t_w \leqslant 2.24\sqrt{E/F_y} & \left\{ 1.0 \quad (\Phi=1.00)\ (\Omega=1.50) \right. \\ \text{other I shape members and channels sections} & \left\{ \begin{array}{ll} 1.0 \quad (\Phi=0.90)\ (\Omega=1.67) & \text{for } h/t_w \leqslant 1.10\sqrt{K_v.E/F_y} \\ \dfrac{1.10\sqrt{K_v.E/F_y}}{h/t_w} \quad (\Phi=0.90)\ (\Omega=1.67) & \text{for } h/t_w > 1.10\sqrt{K_v.E/F_y} \end{array} \right. \end{cases}$$

$$K_V = \begin{cases} 5.34 & \text{without web stiffener} \\ 5+\dfrac{5}{(a/h)^2} & \text{with web stiffener and } a/h \leqslant 3 \\ 5.34 & \text{with web stiffener } a/h > 3 \end{cases}$$



a: clear distance between transverse stiffeners.

MINOR AXIS SHEAR IN DOUBLY SYMMETRIC AND SINGLY SYMMETRIC SHAPES:

$$K_V = 1.2$$

$$V_n = 0.6F_y\ b_f\ t_f\ C_{V2}$$

$$C_{V2} = \begin{cases} 1.0 & \text{for } h/t_w \leqslant 1.1\sqrt{K_V.E/F_y} \\ \dfrac{1.1\sqrt{K_V.E/F_y}}{h/t_w} & \text{for } 1.1\sqrt{K_V.E/F_y} < h/t_w \leqslant 1.37\sqrt{K_V.E/F_y} \\ \dfrac{1.51E.K_V}{(h/t_w)^2.F_y} & \text{for } h/t_w > 1.37\sqrt{K_V.E/F_y} \end{cases}$$

$$(h/t_w) = \begin{cases} b_f/2t_f & \text{I shaped \& tees} \\ b_f/t_f & \text{channels} \end{cases}$$

8-COMBINED FORCES (AISC 360-CHAPTER H)

DOUBLY AND SINGLY SYMMETRIC MEMBERS

1) COMPRESSION + DOUBLE AXIS FLEXURE

$$\text{for } \frac{Pr}{Pc} \geqslant 0.2 \Longrightarrow \frac{Pr}{Pc} + \frac{8}{9}\left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leqslant 1.0$$

$$\text{for } \frac{Pr}{Pc} < 0.2 \Longrightarrow \frac{Pr}{2Pc} + \left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leqslant 1.0$$

$Pc=\Phi_c\, Pn$ $\quad Mc=\Phi_b\, Mn$ $\quad Pr,Mr$: required strengths

2) TENSION + DOUBLE AXIS FLEXURE

$$\text{for } \frac{Pr}{Pc} \geqslant 0.2 \Longrightarrow \frac{Pr}{Pc} + \frac{8}{9}\left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leqslant 1.0$$

$$\text{for } \frac{Pr}{Pc} < 0.2 \Longrightarrow \frac{Pr}{2Pc} + \left(\frac{Mrx}{Mcx} + \frac{Mry}{Mcy}\right) \leqslant 1.0$$

$Pc=\Phi_t\, Pn$ $\quad Mc=\Phi_b\, Mn$ $\quad Pr,Mr$: required strengths

Cb may be increased by $\sqrt{1+\frac{Pu}{Pey}}$ $\qquad Pey=\frac{\pi^2 E.Iy}{L_b^2}$

3) COMPRESSION + SINGLE AXIS FLEXURAL

For out of plane and lateral torsional buckling of doubly symmetric rolled compact members subject to single–axis flexure and compression

$$\frac{Pr}{Pcy}\left(1.5-0.5\frac{Pr}{Pcy}\right) + \left(\frac{Mrx}{Cb\ Mcx}\right)^2 \leqslant 1.0 \text{(SEE H.1.3)}$$

9-1-WELDED CONNECTIONS(AISC 360-CHAPTER J)

1) GROOVE WELD : .... (CJP)

$R_n = F_{nw}.A_{we}$

weld strength (F_{nw}) is taken equal to base metal strength in any kind of stresses.

2) FILLET WELD :

$R_n = F_{nw}.A_{we}$ .... $(\Phi=0.75)(\Omega=2.00)$

$F_{nw} = 0.6F_{EXX}$

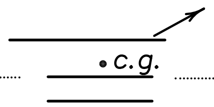
$A_{we} = 0.707L.w$ effective area of the weld

w: size of weld leg.

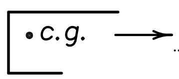
NOTES:

1) for end loaded fillet weld effective weld length is $\beta.L$ where $\beta = 1.2-0.002(L/w) \leqslant 1.0$, $\beta \geqslant 0.6$

2) alternatively we can use the following provisions for fillet weld strength.

linear weld group .... 

.... $Fw = 0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$

group of transverse and longitudinal weld .... 

.... $Rn = max\begin{cases} R_{nwl}+R_{nwt} \\ 0.85R_{nwl}+1.5R_{nwt} \end{cases}$

eccentrically in plane loaded group of weld .... refer to code for details .... $\begin{cases} Rnx = \sum Fwix.Awi \\ Rny = \sum Fwiy.Awi \end{cases}$

θ: angle between the line of action of the required force and the weld longitudinal axis , degrees

9-2-BOLTS-BEARING TYPE *(AISC 360-CHAPTER J)*

1) SHEAR STRENGTH :

$R_n = F_{nv} . A_b$ .... $(\Phi=0.75)\,(\Omega=2.00)$

$$A325 \begin{cases} F_{nv}=3720 \ (N) & Kg/Cm^2 \\ F_{nv}=4690 \ (E) & Kg/Cm^2 \end{cases}$$

$$A490 \begin{cases} F_{nv}=4690 \ (N) & Kg/Cm^2 \\ F_{nv}=5790 \ (E) & Kg/Cm^2 \end{cases}$$

$$THREADED\ PART \begin{cases} F_{nv}=0.45Fu \ (N) \\ F_{nv}=0.563Fu \ (E) \end{cases}$$

E =threads are excluded from shear planes

N =threads are not excluded from shear planes

2) TENSILE STRENGTH :

$R_n = F_{nt} . A_b$ .... $(\Phi=0.75)\,(\Omega=2.00)$

$A325 \text{ ----> } F_{nt}=6200\ Kg/Cm^2$

$A490 \text{ ----> } F_{nt}=7800\ Kg/Cm^2$

$THREADED\ PART \text{ ----> } F_{nt}=0.75Fu$

minimum edge distance (mm)

	SH.	R.TH.
M16	28	22
M20	34	26
M22	38	28
M24	42	30
M27	48	34
M30	52	38
M36	64	46
over	1.75d	1.25d

3) COMBINED TENSION AND SHEAR :

$R_n = F'_{nt} . A_b$ .... $(\Phi=0.75)\,(\Omega=2.00)$

$$F'_{nt}=1.3\,F_{nt} - \frac{F_{nt}}{\Phi F_{nv}} f_v \leqslant F_{nt}$$

F'_{nt}: *modified nominal tensile strength*

4) BEARING AND TEAROUT STRENGTH AT BOLT HOLES:

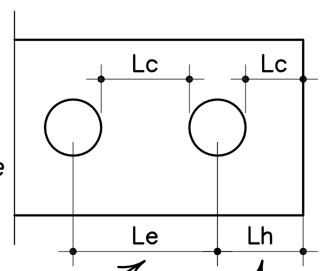
for standard,oversized and short slotted holes

1–Bearing

$$Rn= \begin{cases} 2.4d.t.Fu & \text{deformation at the bolt hole at service load is a design consideration} \\ 3d.t.Fu & \text{otherwise} \end{cases}$$

2–Tearout $(\Phi=0.75)\,(\Omega=2.00)$

$$Rn= \begin{cases} 1.2L_c\,t.Fu & \text{deformation at the bolt hole at service load is a design consideration} \\ 1.5L_c\,t.Fu & \text{otherwise} \end{cases}$$



$max=12t_{min}<150mm$

$min=3d$

$max=24t_{min}<305mm$ *no corrosion*

$max=14t_{min}<180mm$ *corrosion*

9-3-BOLTS-SLIP CRITICAL (AISC 360-CHAPTER J)

1) SHEAR STRENGTH :

$$R_n = \mu D_u h_f T_b n_s$$

$\Phi = \begin{cases} 0.70 & \text{For long-slotted holes} \\ 0.10 & \text{For standard size and short-slotted holes perpendicular to the direction of the load} \\ 0.85 & \text{For oversized and short-slotted holes parallel to the direction of the load} \end{cases}$

$\mu = \begin{cases} 0.3 & \text{class 'A' surface(mill scale)} \\ 0.50 & \text{class 'B' surface(sand blast)} \end{cases}$

$Du = 1.13 \ldots \dfrac{\text{installed pretension}}{\text{specified pretension}}$

$h_f = \begin{cases} 1.00 & \text{For one filler between connected parts} \\ 0.85 & \text{For two or more fillers between connected parts} \end{cases}$

T_b: minimum pretension load

n_s: number of slip planes

h_f: factor for fillers

BOLTS PRETENSION
Tb (TON)

	A325	A490
M16	9.1	11.4
M20	14.2	17.9
M22	17.6	22.1
M24	20.5	25.7
M27	26.7	33.4
M30	32.6	40.8
M36	47.5	59.5

minimum edge distance (mm)

	SH.	R.TH.
M16	28	22
M20	34	26
M22	38	28
M24	42	30
M27	48	34
M30	52	38
M36	64	46
over	1.75d	1.25d

3) COMBINED TENSION AND SHEAR :

$Rn = K_{sc}(\mu D_u h_f T_b n_s)$ .... available shear strength

$$K_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$$

n_b: number of bolts carrying the applied tension

SH : Shear cut

R.TH : Rolled thermally cut

4) BEARING AND TEAROUT STRENGTH AT BOLT HOLES:

for standard, oversized and short slotted holes

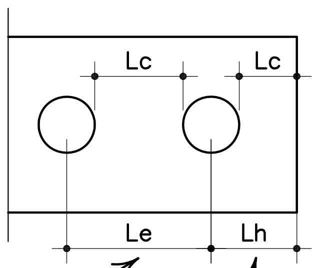
1-Bearing

$Rn = \begin{cases} 2.4d.t.Fu & \text{deformation at the bolt hole at service load is a design consideration} \\ 3d.t.Fu & \text{otherwise} \end{cases}$

2-Tearout

$(\Phi=0.75)\,(\Omega=2.00)$

$Rn = \begin{cases} 1.2L_c t.Fu & \text{deformation at the bolt hole at service load is a design consideration} \\ 1.5L_c t.Fu & \text{otherwise} \end{cases}$



$max = 12t_{min} < 150mm$

$min = 3d$

$max = 24t_{min} < 305mm$ no corrosion

$max = 14t_{min} < 180mm$ corrosion

9-4-CONNECTING ELEMENTS *(AISC 360-CHAPTER J)*

Pu (required strength-LRFD) $\leqslant \Phi Pn$ *(Design strength)*

Pa (required strength-ASD) $\leqslant Pn/\Omega$ *(allowable strength)*

1) TENSION CAPACITY

$$Pn=\min\begin{cases} Fy.Ag & (\Phi=0.90)\ (\Omega=1.67) \quad \text{tensile yielding of connecting elements} \\ Fu.Ae & (\Phi=0.75)\ (\Omega=2.00) \quad \text{tensile rupture of connecting elements} \end{cases}$$

2) SHEAR CAPACITY

$$Rn=\min\begin{cases} 0.6Fy.Agv & (\Phi=1.00)\ (\Omega=1.50) \quad \text{shear yielding of the element} \\ 0.6Fu.Anv & (\Phi=0.75)\ (\Omega=2.00) \quad \text{shear rupture of the element} \end{cases}$$

3) BLOCK SHEAR STRENGTH

$$Rn=\min\begin{cases} 0.6Fu.Anv+Ubs.Fu.Ant \\ 0.6Fy.Agv+Ubs.Fu.Ant \end{cases} \quad (\Phi=0.75)\ (\Omega=2.00)$$

$$An=Ag-\sum d.t+\sum (s^2/4g).t \quad , \quad Ae=U.An \quad , \quad d=\text{Bolt Diameter}+3\text{ mm}$$

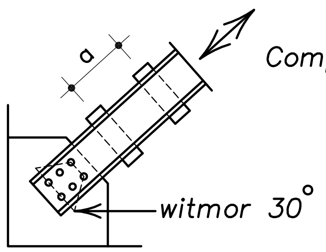
4) COMPRESSION CAPACITY

$$(\Phi=0.90\ ,\ \Omega=1.67)$$

$$\begin{cases} Pn=Fy.Ag & \text{IF } KL/r<25 \\ \text{See Summary Of Compression Members} & \text{IF } KL/r\geqslant 25 \end{cases} \qquad \begin{cases} a/r\leqslant 0.4(KL/r) & \text{SCBF} \\ a/r\leqslant 0.75(KL/r) & \text{Other Systems} \end{cases}$$

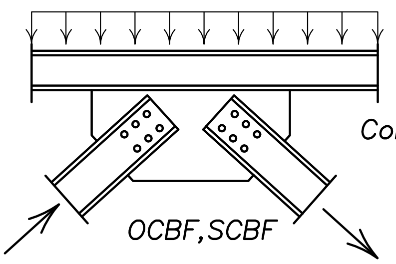
$$\begin{cases} KL/r\leqslant 4\sqrt{E/Fy} \ (117) & \text{for OCBF} \\ KL/r\leqslant 200 & \text{for SCBF} \end{cases}$$

L: Brace Length



$$\text{Compression}\begin{cases} \text{OCBF: } \min\begin{cases} Ry.Fy.Ag \\ 1.1Fcre.Ag \end{cases} \\ \text{SCBF: } \min\begin{cases} Ry.Fy.Ag \\ 1.14Fcre.Ag \end{cases} \end{cases}$$

Tension: Ry.Fy.Ag(OCBF,SCBF)



$$\text{Compression}\begin{cases} \text{OCBF: } 0.3Pn \qquad Pn=Fcr.Ag \\ \text{SCBF: } \min\begin{cases} 0.3Ry.Fy.Ag \\ 0.3\text{x}1.14Fcre.Ag \end{cases} \end{cases} \qquad \text{Tension}\begin{cases} \text{OCBF: MIN}\begin{cases} Ry.Fy.Ag \\ \text{Amplified seismic load} \end{cases} \\ \text{SCBF: } Ry.Fy.Ag \end{cases}$$

NOTES:

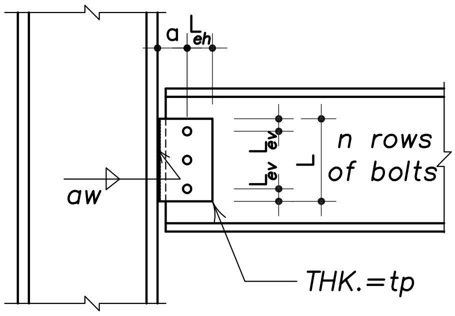
1) *Ubs=1.00 for uniform tension, Ubs=0.5 for non uniform tension*

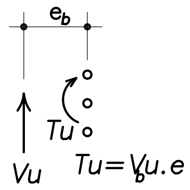
2) *For bolted splice plates Ae=An<0.85Ag*

9-5-FIN PLATE CONNECTION

(AISC STEEL CONSTRUCTION MANUAL 14th, 2011-CHAPTER 10)

Conventional Configuration





Limitations :

1-Only a single vertical row of bolts is permitted.

2-The number of bolts (n) in the connection, must be between 2 and 12.

3-The distance from the bolt line to the weld line (a) must be equal to or less than 90mm.

4-Standard holes (STD) or short-slotted holes transverse to the direction of the supported member reaction (SSLT) are permitted to be used.

5-The vertical edge distance (L_{ev}) satisfy AISC specification table J3.4 requirements.

6-$L_{eh} > 2d$

7-Either the plate thickness, tp, or the beam web thickness, tw, must satisfy the maximum thickness requirement given in below Table.

t_p=*the plate thickness.*

t_w=*the beam web thickness.*

d=the bolt diameter.

n	hole type	e.mm	Maximum tp or tm mm
2 to 5	SSLT	a/2	None
	STD	a/2	d/2+1.6
6 to 12	SSLT	a/2	d/2+1.6
	STD	a	d/2−1.6

ECCENTRICITY :

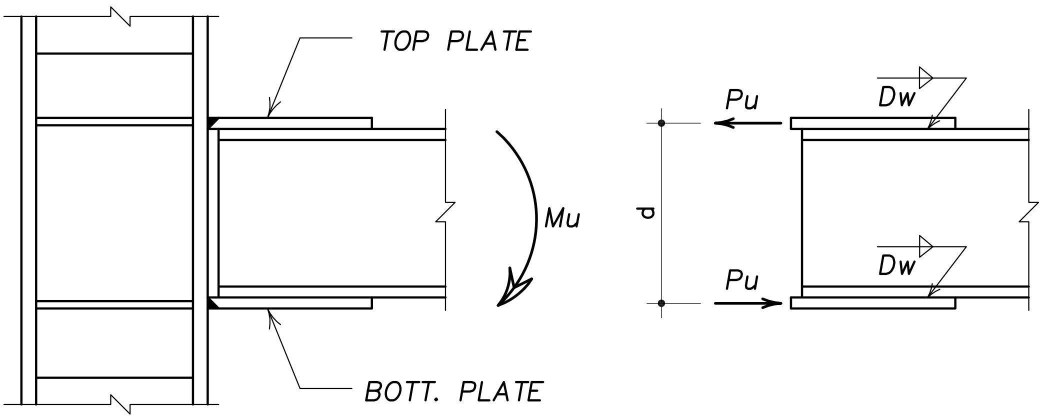
The bolts and plate must be checked for required shear with an eccentricity equal to e. Plate buckling will not control for the conventional configuration.

WELD SIZE :

$aw = (5/8).t_p$ .... *for E70 electrode*

when the dimensional and other limitations of the conventional method are not satisfied extended configuration can be used.

9-6-WELDED RIGID CONNECTION (AISC 360-CHAPTER 2)



TOP PLATE :

Pu=Mu/d

plate area   Ap=Pu/0.9Fy

weld length   Lw=Pu/0.75(0.6FExx)x0.707Dw

CJP weld need not be checked.

BOTTOM PLATE :

Pu=Mu/d

plate area   Ap=Pu/0.9Fy

weld length   Lw=Pu/0.75(0.6FExx)x0.707Dw

CJP weld need not be checked.

CHAPTER J10 CONTROL :

1– flange local bending

2– web local yielding

3– web crippling

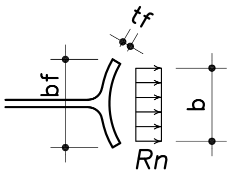
5– web compression buckling

6– web panel zone shear

8– stiffener requirements

9-7-CONCENTRATED FORCE (AISC 360-CHAPTER J)

1) FLANGE LOCAL BENDING : $(\Phi=0.90)\ (\Omega=1.67)$



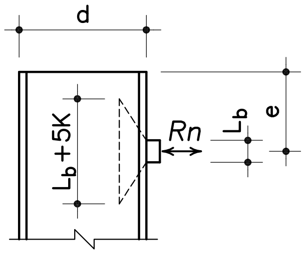
if $b < 0.15bf$ need not to be checked

$$Rn=\begin{cases} 6.25t_f^2.Fyf & \text{far from end} \quad \text{for } e \geqslant 10tf \\ 6.25t_f^2.Fyf/2 & \text{near to end} \quad \text{for } e < 10tf \end{cases}$$

half depth stiffener if necessary

e: the distance from member end.

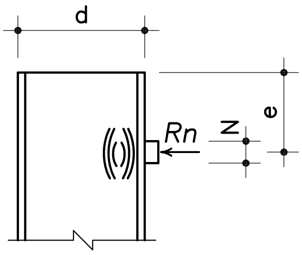
2) WEB LOCAL YIELDING : $(\Phi=1.00)\ (\Omega=1.50)$

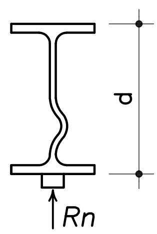


half depth stiffener or a doubler plate if necessary

$$Rn=\begin{cases} (5K+L_b)Fyw.tw & \text{far from end} \quad \text{for } e > d \\ (2.5K+L_b)Fyw.tw & \text{near to end} \quad \text{for } e \leqslant d \end{cases}$$

3) WEB CRIPPLING : $(\Phi=0.75)\ (\Omega=2.00)$





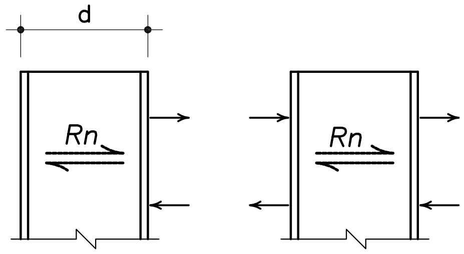
half depth stiffener if necessary

$$Rn=\begin{cases} 0.80t_w^2\left[1+3\left(\frac{L_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E.Fyw.t_f}{t_w}}\,Q_f & \text{far from end} \quad \text{for } e \geqslant d/2 \\ 0.40t_w^2\left[1+3\left(\frac{L_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E.Fyw.t_f}{t_w}}\,Q_f & \text{near to end, small sitting} \quad \text{for } e < d/2,\ L_b/d \leqslant 0.2 \\ 0.40t_w^2\left[1+\left(\frac{4L_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E.Fyw.t_f}{t_w}}\,Q_f & \text{near to end, large sitting} \quad \text{for } e < d/2,\ L_b/d > 0.2 \end{cases}$$

$Q_f=1$ for wide-flange sections.

9-8-CONCENTRATED FORCE (AISC 360-CHAPTER J)

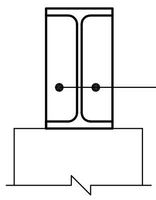
6) WEB PANEL ZONE SHEAR : $(\Phi=0.90)\ (\Omega=1.67)$



$$R_n = \begin{cases} \text{NOT considering panel zone deformation in analysis} & \begin{cases} 0.60F_y.d_c.t_w & \text{for } P_r \leqslant 0.4P_y \\ 0.60F_y.d_c.t_w\left(1.4-\dfrac{P_r}{P_y}\right) & \text{for } P_r > 0.4P_y \end{cases} \\ \text{considering panel zone deformation in analysis} & \begin{cases} 0.60F_y.d_c.t_w\left(1+\dfrac{3b_{cf}.t_{cf}^2}{d_b.d_c.t_w}\right) & \text{for } P_r \leqslant 0.75P_y \\ 0.60F_y.d_c.t_w\left(1+\dfrac{3b_{cf}.t_{cf}^2}{d_b.d_c.t_w}\right)\left(1.9-\dfrac{1.2P_r}{P_y}\right) & \text{for } P_y > 0.75P_y \end{cases} \end{cases}$$

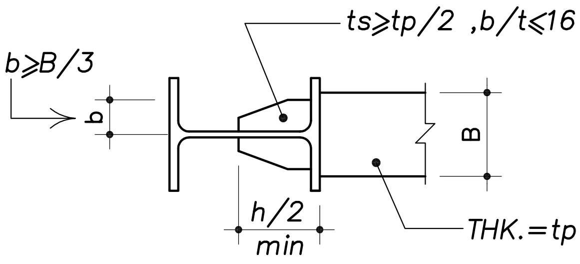
P_r : required axial strength of the column

7) UNFRAMED ENDS OF BEAMS :



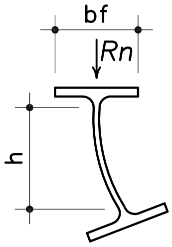
a pair of full depth stiffener required

8) STIFFENER REQUIREMENTS :



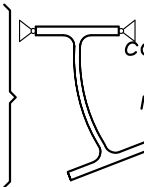
9-9-CONCENTRATED FORCE (AISC 360-CHAPTER J)

4) WEB SIDESWAY BUCKLING : (Φ=0.85) (Ω=1.76)

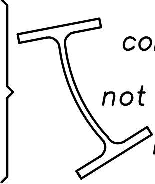


bottom flange lateral brace if necessary

$$Rn=\begin{cases}\begin{cases}\dfrac{C_r\,.\,t_w^3\,.\,t_f}{h^2}\left[1+0.4\left(\dfrac{h/t_w}{L_b/b_f}\right)^3\right] & \text{for } \dfrac{h/t_w}{L_b/b_f}\leqslant 2.3\\[2ex] \text{buckling mode does not apply} & \text{for } \dfrac{h/t_w}{L_b/b_f}> 2.3\end{cases}\\[6ex] \begin{cases}\dfrac{C_r\,.\,t_w^3\,.\,t_f}{h^2}\left[0.4\left(\dfrac{h/t_w}{L_b/b_f}\right)^3\right] & \text{for } \dfrac{h/t_w}{L_b/b_f}\leqslant 1.7\\[2ex] \text{buckling mode does not apply} & \text{for } \dfrac{h/t_w}{L_b/b_f}> 1.7\end{cases}\end{cases}$$



if compression flange restrained about rotation

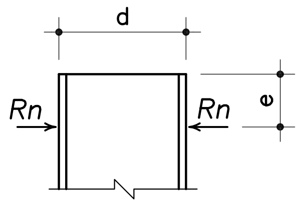


if compression flange not restrained about rotation

$l=max(l_t\,,l_b)$ $l_t\,,l_b$: top and bottom flange unbraced length

$$C_r=\begin{cases}6.73\text{x}10^7 \ \ Kg/Cm^2 & \text{for } Mu < My\\ 3.37\text{x}10^7 \ \ Kg/Cm^2 & \text{for } Mu \geqslant My\end{cases}$$

5) WEB COMPRESSION BUCKLING : (Φ=0.90) (Ω=1.67)

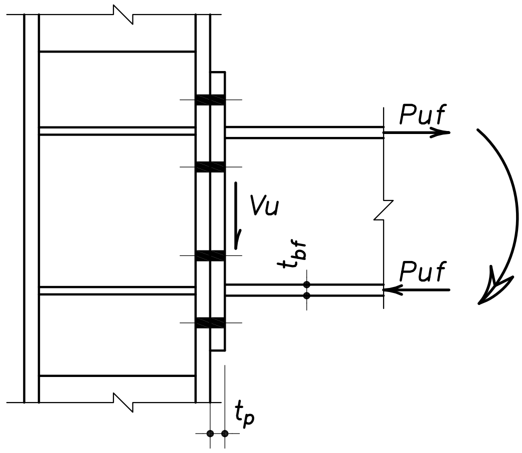


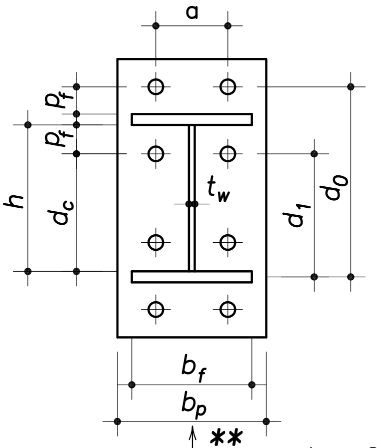
full depth stiffener if necessary

$$Rn=\begin{cases}\dfrac{24t_w^3\sqrt{E.Fyw}}{h}\,Q_f & \text{far from end} \quad \text{for } e \geqslant d/2\\[2ex] \dfrac{12t_w^3\sqrt{E.Fyw}}{h}\,Q_f & \text{near to end} \quad \text{for } e < d/2\end{cases}$$

$Q_f=1$ for wide-flange sections.

10- 4 BOLT END PLATE CONNECTION (AISC 358-CHAPTER 6)





END PLATE THICKNESS :

$$S_h = \begin{cases} \min(d/2, 3b_f) & \text{unstiffened connection} \\ L_{st} + t_p & \text{stiffened connection} \end{cases}$$

$$M_{pr} = C_{pr} R_y F_y Z_e$$

$$M_f = M_{pr} + V_u S_h$$

$V_{gravity}$=resulting from $1.2D + f_1 L + 0.2S$

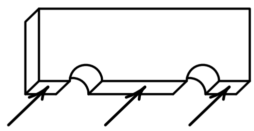
$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity}$$

$f_1 \geqslant 0.50$

$$t_{p\ req.} = \sqrt{\frac{1.11 M_f}{\underset{\uparrow 1}{\Phi_d} F_{yp} . Y_p}}$$ end plate thickness

Y_p =end-plate yield line mechanism parameter from Tables 6.2, 6.3 or 6.4, in. (mm)

END PLATE SHEAR CHECK :



$$R_u = P_{uf}/2$$

REQUIRED BOLT DIAMETER:

$$d_{b,req} = \sqrt{\frac{2M_f}{\pi \Phi_d F_{nt} . (h_0 + h_1)}}$$

BOLT SHEAR RUPTURE : $V_u < \Phi_n R_n = \Phi_n (nb) F_{nv} A_b$

bolt-bearing/tear-out

failure of the end-plate and column flange

ni: number of inner bolts. $V_u < \Phi_n R_n = \Phi_n (ni) r_{ni} + \Phi_n (no) r_{no}$

$r_{ni} = \min(1.2 L_C\, t\, F_u, 2.4 d_b\, t\, F_u)$

no: number of outer bolts.

$r_{no} = \min(1.2 L_C\, t\, F_u, 2.4 d_b\, t\, F_u)$

WELD OF BEAM WEB :

NEAR TO TENSION BOLTS : $D_w = 0.9 F_y . t_w / (2 \times 0.707 \times 0.75 \times 0.6 F_{EXX})$

SHEAR CHECK : $\underset{\uparrow}{L_w} = \min(h/2, d_c - 2d_b)$

effective weld length

$$R_u = V_u \quad , \quad \Phi R_n = 0.75 \times 0.707 \times 0.6 F_{EXX} . L_w . D_w$$