*** DESIGN CALCULATIONS COVER SHEET ***

SDS/2 v 6.322

SYMBOLS

A	Section gross area or Dimension from weld line to C/L bolts for a single-plate shear connection
Ab	Bolt area based on nominal diameter
Ae	Effective net area
Afg	Gross beam flange area
Afn	Net beam flange area
Ag	Gross area
Agt	Gross tension area
Agv	Gross shear area
An	Net area
Ant	Net tension area
Anv	Net shear area
Ast	Cross sectional area of a pair of stiffeners
At	Net tension area
Av	Net shear area
B	Allowable tension per bolt, AISC Page 4-90
Bc	Bolt tension including prying action
Bf	Section flange width
C	Fastener or weld group coefficient (e.g. TABLE XI, XIX) ultimate strength method unless noted.
Ca	Coefficient used in moment end PL design
Cc	Slenderness ratio separting elastic & inelastic buckling
Cdb	Bottom cope depth
Cdt	Top cope depth
CJP	Complete joint penetration -- weld
Clb	Bottom cope length
Clip_stbk	Dim. from face of clip angle to end of beam
Clt	Top cope length
Col_spa	Spacing between bolt columns
Column	Number of fasteners perpendicular to the line of force
Cn_depth	Connection element depth
Cn_thick	Connection element thickness
Cn_width	Connection element width
Cv	Ratio of critical web stress to the shear yield stress of the web material
D	Section depth
Db	Nominal bolt diameter
Dc	Depth of column web clear of fillets
Dh	Hole dimension: Db + 1/16 (2 mm), ASD shear limit state Db + 1/8 (4 mm), ASD tension limit state. Nominal dim of bolt hole + 1/16, LRFD

E	Modulus of elasticity of steel 29,000 KSI (200,000 MPa)
Eff_weld	Max. effective weld size based on mtrl thickness, <= actual weld size
Et	Composite slab eff. width MIN[Ln/4, Bm spacing] (I1.a,b)
Fa	Allowable axial stress
Fb	Allowable bending stress
Fbt	A325 or A490 bolt material min. tensile stress
Fbu	A325 or A490 bolt material min. yield stress
Fcr	Critical stress
Fil_rad	Rolled section fillet radius, K_dist - Tf
Fn	Nominal shear rupture strength
Ft	Allowable tensile stress
Fp	Allowable bearing stress, calculated using J3.7,8,9 for bolts or J8 for contact surfaces.
Fu	Specified material minimum tensile stress
Fv	Allowable shear stress
Fw	Nominal weld strength, talbe J2.5
Fexx	Nominal specified tensile strength of weld metal
Fr	Allowable force on a weld
Fy	Specified Material minimum yield stress
H	Width of HSS section connecting to a gusset plate
Hb	Horiz. interface force between gusset & beam
Hc	Horiz. interface force between gusset & col.
Ip	Polar moment of inertia
Ix	Moment of inertia
Ix_net	Net moment of inertia
K	Effective length factor
K_dist	Rolled section (k) dist., flg. to toe of fillet
K1_dist	W shape, C/L web to toe of fillet
La	Lever arm from load to C.G. of bolt or weld group
Lb	Unbraced length of section or elememt
Le	Distance from free edge to center of bolt in the direction of the line of transmitted force.
Leff	Effective web length, stiffener design
Leg	Leg size against beam web, clip angle connection or leg size against gusset for angle brace
Lh	Horizontal edge distance from fastener C/L
Ln	Beam span length
Lv	Vertical edge distance from fastener C/L
Mb	Moment at brace gusset and beam interface
Mc	Moment at brace gusset and col. interface
Min_brg	Calculated minimum beam bearing length
Mn	Nominal flexural strength
Moment	Beam end moment, (+) = clockwise beam end rotation
Mp	Beam plastic moment strength
Mr	Beam elastic moment strength
Mu	Required flexural strength
N	Length of bearing

Osl	Outstanding angle leg size
Pair	Pairs of transverse beam stiffeners, 1 or 2
Pbf	Factored flange or connection plate force; computed flange force multiplied by a load factor
PHI	LRFD resistance factor
PI	3.14159.....
Pn	Nominal axial strength
Pu	Required tensile or compressive strength
Q	Prying force per bolt at design load, AISC pg 4-90
Qs	Stress reduction factor, Appendix B
Rbs	Resistance to block shear
Rn	Nominal strength
Ro	Shear tab or tee strength at yield
Row	Number of fasteners in the line of force
Rstr	Bolt slip resistance, (A-J3-1)
Rv	Force transmitted by one fastener
Setback	Dist from face of support to end of beam
Shear	Number of shear planes: (1 = single shear; 2 = double shear)
Sn	Section modulus of the coped portion of a beam
Spa or S	Bolt spacing
SQR[ ]	Square of expression in brackets
SQRT[ ]	Square root of expression in brackets
Sx	Section modulus
Sx_net	Net section modulus
S_g	S^2/4g, AISC section B2
T_allow	Allowable tension per bolt
Tb	A325 or A490 bolt pre-tension: ASD Table J3.7 LRFD Table J3.1
Tc	HSS brace welded pl end fitting, cap plate thickness
Tf	Flange or angle leg thickness
Tw	Web or tube wall thickness
Ts	HSS brace welded pl end fitting, tee stem thickness
T_slab	Concrete slab thickness, composite design
T_sup	Thickness of supporting member
U	Shear lag reduction coefficient, Chapter B3 or HSS manual, specification section 2.1
Vb	Vert. force at gusset-beam interface
Vc	Vert. force at gusset-column interface
Vn	Nominal shear strength
Weld_size	Fillet weld leg size or groove weld throat
Weld_len	Length of weld
Weld_spa	Spacing between two parallel weld segments
Wg	Element gross width
Wn	Element net width
Ws	'Whitmore' section width, gusset Pl design
Zx	Plastic section modulus
Ze	Effective plastic section modulus

a	Clear dist. between transverse stiffeners
b	Width of compression element, or other part
c	Cope length from end of beam web
d	Depth of connected element
dc	Beam cope depth
e	Cope length from face of conn. or weld line
eb	Shear tab bolt design eccentricity
ew	Shear tab weld design eccentricity
fa	Calculated axial stress
fb	Calculated bending stress
fp	Calculated bearing stress
fr	Calculated force on a weld
ft	Calculated tension stress
fv	Calculated shear stress
f'c	Compression strength of concrete
g	Angle leg gage
g1,g2	Angle leg gages, 5 in or larger leg size
h	Clear dist. between flanges of a beam or girder.
ho	Depth of the remaining web at a coped beam section
kl	Horiz. weld segment length, Table XXII
kv	Shear buckling coefficient for girder webs
l	Length
n	Modular ratio Es/Ec, 9
r	Radius of gyration
t	Thickness of connected element
tc	Thickness of connected element required to develop 'B' in bolts with no prying action, ASD AISC pg 4-90

Manual of Steel Construction, LRFD Third Edition.

Load and Resistance Factor Design Specification for
Structural Steel Buildings, December 27, 1999

Hollow Structural Sections Connections Manual, 1997

TENSION MEMBERS

Gross section yielding:
PHI = .9
Pn = Fy * Ag (D1-1)
Design strength = PHI * Pn

Net section fracture:
PHI = .75
Pn = Fu * Ae (D1-2)
Design strength = PHI * Pn

COMPRESSION MEMBERS

Width-thickness ratios < Lambda_r from B5.1:
PHI = .85
Design strength = PHI * Pn
Pn = Ag * Fcr

For Lambda_c <= 1.5,
Fcr = (.658)^(SQR[Lambda_c]) * Fy (E2-2)

For Lambda_c > 1.5,
Fcr = [.877 / SQR[Lambda_c]] * Fy (E2-1)

Lambda_c = Kl/(r*PI) * SQRT[Fy / E] (E2-4)

Width-thickness ratios > Lambda_r from B5.1:
Reduction factor Qs:

Angles:
76/SQRT[Fy] < b/t < 155/SQRT[Fy]:
Qs = 1.34 - .00447 * (b/t) * SQRT[Fy] (A-B5-3)
b/t >= 155/SQRT[Fy]:
Qs = 15500 / (Fy * SQR[b/t]) (A-B5-4)

Plates:
95/SQRT[Fy] < b/t < 176/SQRT[Fy]:
Qs = 1.415 - .00437 * (b/t) * SQRT[Fy] (A-B5-5)
b/t >= 176/SQRT[Fy]:
Qs = 20000 / (Fy * SQR[b/t]) (A-B5-4)

Stems of tees:
127/SQRT[Fy] < b/t < 176/SQRT[Fy]:
Qs = 1.908 - .00715 * (b/t) * SQRT[Fy] (A-B5-6)
b/t >= 176/SQRT[Fy]:
Qs = 20000 / (Fy * SQR[b/t]) (A-B5-7)

BMS AND FLEXURAL MEMBERS

Yielding: PHI = .9
Mn = Mp = Fy * Z (F1-1)
Design strength = PHI * Mn

Weak axis bending/bars: Mr = Fy * Sx (F1-11)
Design strength = PHI * Mr

Shear strength: PHI = .9
Vn = nominal shear strength

h/Tw <= 418/SQRT[Fy]
Vn = .6 * Fy * Aw (F2-1)
h/Tw <= 535/SORT[Fy]
Vn = .6Fy*Aw(418/SQRT[Fy] / (h/Tw) (F2-2)
h/Tw <= 260
Vn = 132000*Aw/SQR[h/Tw] (F2-3)
Design Strength = PHI * Vn

COMBINED FORCES:

Pu/(PHI*Pn) >= .2
Pu/(PHI*Pn) + 8/9 (Mu / (PHI*Mn) <= 1 (H1-1a)

Pu/(PHI*Pn) < .2
Pu/(2*PHI*Pn) + 8/9 (Mu / (PHI*Mn) <= 1 (H1-1b)

WELDS J2.4

Fillet welds: PHI = .75
Fw = .6 * Fexx

Linear weld group loaded in-plane through the center of gravity:
Fw = .6Fexx(1.0 + .5 (SIN[Theta])^1.5) AJ2.4

CJP welds: PHI = .9
Fw = Fy

Design strength = PHI * Fw * Aw

FASTENERS J2.4

Tensile strength: PHI = .75
Design Strength = PHI * Fn * Ab
(Fn = Tensile strength from table J3.2)

Shear strength:
Brg type connectors PHI = .75
Fn = Value from table J3.2
Design strength = PHI * Fn * Ab

```
        SC at factored load   Rstr = 1.13 * mu * Tm          AJ3.8b
                              Tm min fastener tension, table J3.1
                              mu = .33 for class A surfaces
                              mu = .50 for class B surfaces
                              mu = .35 for class C surfaces
                              PHI = 1.0 for STD holes
                              PHI = .85 for OVS * SSL holes
                              PHI = .75 for LSLT holes
                              PHI = .60 for LSLP holes
                              Design strength = PHI * Rstr

      Bearing strength at bolt holes
        STD, SSL, OVS hole types:
                     Rn = 1.2*Lc*t*Fu <= 2.4 * d*t*Fu (J3-2a)
        LST hole type:
                     Rn = 1.0*Lc*t*Fu <= 2.* d*t*Fu   (J3-2c)
        PHI = .75
        Design strength = PHI * Rn

      Combined tension and shear, brg type bolts:
              PHI = .75, Design strentgh = PHI*Ft*Ab
              Ft = value from TABLE J3.5

      Combined tension and shear, slip-critical bolts
      designed at factored loads:
       Rstr = 1.13*mu*Tm * [1- Tu / (1.13 * Tm ]  (A-J3-2)

  SHEAR RUPTURE STRENGTH:
       PHI = .75
       Rn = .6*Fu*Anv                                     (J4-1)
       Design strength = PHI * Rn

  TENSION RUPTURE STRENGTH:
       PHI = .75
       Rn = Fu*Ant                                        (J4-2)
       Design strength = PHI * Rn

  BLOCK SHEAR RUPTURE STRENGTH:
       PHI = .75
       Rn = MAX[(.6FyAgv + FuAnt), (.6FyAnv + FyAgt) ]
                      (J4-3a)            (J4-3b)
       Design strength = PHI * Rn

  TENSION YIELDING OF A CONNECTING ELEMENT:
       PHI = .9
       Rn = AgFy                                         (J5-1)
       Design strength = PHI * Rn

  TENSION RUPTURE OF A CONNECTING ELEMENT:
       PHI = .75
       Rn = AnFu                                         (J5-2)
       Design strength = PHI * Rn
```

SHEAR YIELDING OF A CONNECTING ELEMENT:

PHI = .9
Rn = .6AgFy (J5-3)
Design strength = PHI * Rn

BEARING STRENGTH:

PHI = .75
Rn = 1.8FyApb (J8-1)
Design strength = PHI * Rn

LOCAL FLANGE BENDING:

PHI = .9
Rn = 6.25 SQR[Tf]Fy (K1-1)
Design strength = PHI * Rn

LOCAL WEB YIELDING:

PHI = 1.0
Compression force applied at a distance from the member end > the depth of the member,
Rn = (5k + N)FyTw (K1-2)

Compression force applied at a distance from the member end <= the depth of the member,
Rn = (2.5k + N)FyTw (K1-3)

Design strength = PHI * Rn

LOCAL WEB CRIPPLING:

PHI = .75
Compressive force is applied at a distance from the member end >= d/2: Formula(K1-4)

Rn = 135Tw^2[1 + 3N/d * (Tw/Tf)^1.5] * (FyTf/Tw)^.5

Compressive force is applied at a distance from the member end < d/2:
N/d <= .2 Formula(K1-5a)

Rn = 68Tw^2[1 + 3N/d * (Tw/Tf)^1.5] * (FyTf/Tw)^.5

N/d > .2 Formula(K1-5b)

Rn = 68Tw^2[1 + (4N/d - .2) * (Tw/Tf)^1.5] * (FyTf/Tw)^.5

Design strength = PHI * Rn

```
  SIDESWAY WEB BUCKLING:
       PHI = .85
       Design strength = PHI * Rn
       Cr = 480000

       Compression flg. restrained against rotation:
       (h/tw)/(l/bf) <= 2.3,
       Rn = Cr Tw^3 Tf/h^2 [1 + .4 ((h/tw)/(l/bf))^3   (K1-6)

       Compression flg. not restrained against rotation:
       (h/tw)/(l/bf) <= 1.7,
       Rn = Cr Tw^3 Tf/h^2 [.4 ((h/tw)/(l/bf))^3       (K1-7)

  WEB COMPRESSION BUCKLING:
       PHI = .9
       Rn = 4100 * CUBE[Tw] *SQRT[FY]/h                 (K1-8)
       When the compressive force is applied at a
       distance from the member end < .5d, Rn is
       by 50 percent.
       Design strength = PHI * Rn

  PANEL-ZONE WEB SHEAR:
             PHI = .9
       For Pu <= .4Py
             Rv = .6Fy * dc * Tw                        (K1-9)

       For Pu > .4Py
             Rv = .6Fy * dc * Tw (1.4 - Pu/Py)          (K1-10)
       Design strength = PHI * Rv
```

SETUP RESISTANCE FACTOR SUMMARY:

Section B10-1	Net area	PHI = 0.75
Section B10-1	Gross area	PHI = 0.90
Section D1(a)	Gross section yielding	PHI = 0.90
Section D1(b)	Net section fracture	PHI = 0.75
Sections D3(a) & (b)	Shear eff.	PHI = 0.75
Sections E2, E3	Design compression	PHI = 0.85
Sections F1.1, F1.2	Flexure	PHI = 0.90
Sections F2.2	Shear	PHI = 0.90
Sections H2(a)		PHI = 0.90
Sections H2(b)		PHI = 0.90

Welds, TABLE J2.5

CJP eff area T,C,base metal	PHI = 0.90
CJP eff area shear, weld electrode	PHI = 0.80
PJP Base metal T or C	PHI = 0.90
PJP eff area shear, electrode	PHI = 0.75
PJP eff area normal ten., electrode	PHI = 0.80
Fillet, eff area shear	PHI = 0.75
Fillet, ten or comp, base metal	PHI = 0.90
Plug/slot weld shear, electrode,base metal	PHI = 0.75

Fasteners, TABLE J3.2

Tension or shear strength	PHI = 0.75

Slip-critical connections designed at factored loads

Section J3.8b(a)	STD holes	PHI = 1.00
Section J3.8b(b)	OVS and SSL	PHI = 0.85
Section J3.8b(c)	LSLT (transverse)	PHI = 0.70
Section J3.8b(d)	LSLP (parallel)	PHI = 0.60

Section J3.10	Bearing at bolt holes	PHI = 0.75
Section J4.1	Shear rupture strength	PHI = 0.75
Section J4.2	Tension rupture strength	PHI = 0.75
Section J4.3	Block shear rupture strength	PHI = 0.75
Section J5.2(a)	Tension yielding	PHI = 0.90
Section J5.2(b)	Tension rupture	PHI = 0.75
Section J5.3	Shear yielding	PHI = 0.90

Section J8	Bearing strength	PHI = 0.75
Section J8	Brg. strength on concrete	PHI = 0.60
Section K1.2	Local flange bending	PHI = 0.90
Section K1.3	Local web yielding	PHI = 1.00
Section K1.4	Local web crippling	PHI = 0.75
Section K1.5	Sidesway web buckling	PHI = 0.85
Section K1.6	Web compression buckling	PHI = 0.90
Section K1.7	Panel zone shear	PHI = 0.90

LRFD RESISTANCE (DESIGN STRENGTH) CALCULATIONS

```
PHI = LRFD specification resistance factor

Load = Connection shear strength, PHI*Rn, for the limit
       state being checked.
Moment = Connection moment strength, PHI*Rn, for the limit
         state being checked.
Tension = Connection tension strength, PHI*Rn, for the
          limit state being checked.
Axial load = Brace tension or compression strength, PHI*Rn
             for the limit state being checked.

(Load and Tension are calculated from combined shear
 and axial loads where applicable)
Design formulas are based on Imperial units.

Calculation        Limit state and design strength formulas:
number:


  (1) Bolt shear, eccentricity not considered:
       Bearing type bolts:
         Load = PHI * Fn * Ab * Row * Column * Shear
       Slip-critical bolts at factored loads:
         Load = PHI * Rstr * Row * Column * Shear

  (2) Beam web shear:
         Load = PHI * Vn  (F2-1, -2, -3)

  (3) Bolt shear, eccentricity considered:
       Bearing type bolts:
           Rv = PHI * Fn * Ab * Shear
       Slip-critical bolts at factored loads:
           Rv = PHI * Rstr * Ab * Shear
        Load = C * Rv

  (4) Beam web net shear, coped bolted connection:
        An  = Tw * (D - Row * Dh - Cdt - Cdb)
        Load = PHI * .6 * Fu * An

  (5) Beam web shear, coped welded connection:
        Ag   = Tw * (D - Cdt - Cdb)
        Load = PHI * .6 * Fy * Ag

  (6) Coped beam, PL, resistance to block shear, bolted:
        Rn_a = .6Fy*Agv + Fu*Ant      (J4-3a)
        Rn_b = .6Fu*Anv + Fy*Agt      (J4-3b)
        Fu*Ant >= .6Fu*Anv: Load = PHI * Rn_a
        Fu*Ant <  .6Fu*Anv: Load = PHI * Rn_b

  (7) Coped beam resistance to block shear, welded clip:
        (Volume II, page 3-74, -75)
        Coped top and bottom:
           Load = PHI * .6 * Fy * Cn_depth * Tw
        Coped top or bottom:
           F1 = PHI * .6 * Fy * Cn_depth * Tw
           F2 = PHI * Fu( Leg - Setback )Tw / 2
        Load = F1 + F2
```

```
(8) Beam web shear, welded end PL shear connection:
     Eff_depth = MIN[Cn_depth - 2*Weld_size, D - 2*K_dist]
     Aw    = Tw * Eff_depth
     Load = PHI * Vn       (F2-1, -2, -3)

(9) Bolt brg, dbl L brace, angles on same side of gus.:
     Rn is calculated using (J3-2a) or (J3-2c)
     Lc_edge = Le - .5Dh
     Lc_interior = Spacing - Dh
     N_e = number of edge bolts
     N_i = number of interior bolts

     Axial load = PHI*Rn_edge*N_e + PHI*Rn_int*N_i

(10) Bolt shear, dbl L brace, angles on same side of gus.:
       n = number of bolts in connection
       Two columns --   n = 2 * Row
       Four columns --  n = 4 * Row - 2

     Bearing type bolts:
      Axial load = PHI * Fn * Ab * n * Shear
     Slip-critical bolts at factored loads:
      Axial load = PHI * Rstr * n * Shear

(11) Brace tear-out, dbl L brace, angles on same side of gus.:
      Anv = 2t((S - Dh)*(Row - 1) + Le - .5 * dh)
      Agv = 2t(S(Row -1) + Le)
    Two columns --
      Ant = 2t( Leg - g1 - .5 * Dh )
      Agt = 2t(leg - g1)
    Four columns --
      Ant = 2t(Leg - g1 - g2 - 1.5 * Dh + SQR[.5 * S] / 4.0 / ga2)
      Agt = 2t(Leg - g1)

      Rn1 = .6Fy*Agv + Fu*Ant         (J4-3a)
      Rn1 = .6Fu*Anv + Fy*Agt         (J4-3b)
    Axial load = PHI* MAX[ Rn1, Rn1 ]

(12) Gus tear-out, dbl L brace, angles on same side of Pl:
      Agv = 2t(S(Row -1) + Le)
      Agt = 2t(g1)
     Two columns --
       Ant = (2 * g1  - Dh ) * t
       Anv = 2t*( Le - .5*Dh + (S - Dh)*(row - 1))
     Four columns --
       Ant = (g1 + g2 - Dh ) * 2 * t
       Anv = 2t * ((row - 1) * (S- Dh) + S + Le)

     Rn1 = .6Fy*Agv + Fu*Ant       (J4-3a)
     Rn2 = .6Fu*Anv + Fy*Agt       (J4-3b)
     Axial load = PHI* MAX[ Rn1, Rn2 ]

(13) Net brace ten, dbl L brace, angles on same side of gus.:
      S = longitudinal bolt spa.
      U =.75  two bolt rows, U =.85 more than two rows
      Wg = 2( Leg + Osl - Thickness )
      Two columns --  Wn = Wg - 2 * Dh
      Four columns -- Wn = Wg - 4 * Dh + 2 * S_g
      Ant = Wn * t
      Ae = Ant * U
     Axial tension = PHI * Fu * Ae
```

```
(14) Intersection gusset tension, dbl L brace,
      angles on the same side of gusset:
        Two columns -- add = 0
        Four columns -- add = 2 * S_g
        Wn = (b - Column * Dh + add)
        Ant = t * MIN[ Wn, .85 * b]
        Agt = b * t
      Tension = MIN[ PHI *Fu *, PHI* Fy * Agt ]

(15) Connection gross shear:
     Two side gusset PL, End PL or two side clip L:
       Ag   = 2 * Cn_depth * Cn_thick
      Load = PHI * .6 Fy * Ag

(16) Connection net shear, bent pl, clip angle, splice pl:
        Wn = Cn_depth - Dh * Row
        An = 2 * Cn_thick * Wn
      Load = PHI * .6Fu * Anv

(17) Conn. gross shear; single Pl conn., one side gusset
     or clip angle:
       Ag   = Cn_depth * Cn_thick
      Load = PHI * .6 * Fy * Ag

(18) OSL bending, one side clip angle:
       D = Cn_depth      n = Row
       La = Gosl         b = Spa
       Sx_net = t*D^2/6-b^2*n(n^2-1)t(Dh)/6D, Table 12-1
      Load = PHI * Fu * Sx_net / La    (J5-2)

(19) Bending, net section of gusset/shear plate:
       D = Cn_depth      n = Row   b = Spa
       Sx_net = t*D^2/6-b^2*n(n^2-1)t(Dh)/6D, Table 12-1
      Load = PHI * Fu * Qs * Sx_net / eb   (J5-2)

(20) Bearing on connection, eccentricity considered:
      Rn is calculated using (J3-2a) or (J3-2c)
      Lc_edge = Le - .5Dh
      Lc_interior = Spacing - Dh
      Load = C * PHI*Rn*Shear

(21) Connection net shear:
        k=1 for shear tab, shear tee & thru pl
        k = 2 for end plate, dbl clip angle.
        Net shear --
         Wn = Cn_depth-Dh*Row
         Anv = Cn_thick*Wn
         Rn = PHI * k *  .6 * Fu * Anv
```

```
(22) Weld, shear plate to support:
        (eccentricity not considered)
      (For moment connections to WF col webs, deduct
      two .75 corner clips, Cn_depth = Pl depth  - 1.5)
       Aw = 2 * .707 * Weld_size * Cn_depth
      Load = 2 PHI * Fw * Aw

(23) Weld, two side clip angle to support:
      (Volume II page 2-37)
       La   = Osl
       L    = Cn_depth
       Fr   = PHI * .707*Fw*Weld_size
       K    = (9*La/5/L/L)^2+(1/2/L)^2
      Load = SQRT[Fr^2/K]

(24) Fillet weld stress, shear end Pl to beam web:
       Min_web = .707*Fw*Weld_size*2/Fv_web
       Web_factor = MIN[Tw/Min_web,1]
       Weld_len = Cn_depth-2*Weld_size
       Fr = PHI *.707*Fw*Weld_size
      Load = 2*Fr*Weld_len*Web_factor

(25) Column web crippling, formulas (K1-4) & (K1-5a, -5b):
       N = thick of flange or Pl connected to column
      Load applied at a dist. >= d/2 from top of column,
        Rn =.8Tw^2[1+3(N/d)*(Tw/Tf)^1.5]SQRT[E*Fy*Tf/TS]

      Load applied at a dist. <  d/2 from top of column,
        N/d <= .2
        Rn =.40Tw^2[1+3(N/d)*(Tw/Tf)^1.5]SQRT[E*Fy*Tf/TS]

        N/d > .2
        Rn =.40Tw^2[1+(4N/d - .2)*(Tw/Tf)^1.5]SQRT[E*Fy*Tf/TS]

      Moment = PHI * Rn * .95 * D_beam

(26) Clip angle to beam web weld, ecc. considered
      [ shaped weld:
       Side = 1 or 2 (Connection on 1 or 2 sides of beam)
       La = Face of connection to C.G. weld group
       kl = angle leg - clip_stbk
       Min_web = .707*PHI*Fw*Weld_size*Side/(PHI*.6Fy_web)
       Web_factor = MIN[Tw/Min_web,1]
       N = Weld_size*16
      Load = C*N*Side*Web_factor*Cn_depth

(27) Allowable load for bolts with applied axial tension:
      For shear end plates -- the bolt tension is increased
      due to the effect of eccentricity from the bolt group
      c.g. to the centerline of the beam, Vecc:
       applied T / N + applied T * Vecc / bolt group sx
      where N = the number of bolts.
      (See J3.9 for combined tension and shear)
      Load = PHI * Fn * Ab * Row * Column
      See design notes for 'heavy' clip angles.)
```

(28) Unstiffened seat angle, OSL bending:

For seats with 3.5 <= OSL <= 4, & setback <= .5 the AISC procedure is used, with an added check for OSL shear yielding.
For other cases the following procedure applies.

Reaction is taken at the c/l of the required minimum calculated bearing length.

```
  t = angle thickness
  Ef = .25 + Setback +.5*Min_brg
  Eo = Ef-(t+3/8), Toe of fillet to C/L brg
  Zx = l * t^2 / 4
 Load = PHI * Fy * Zx / Eo
     <= PHI*.6*Ag*Fy, seat shear yielding
```

(29) Unstiffened seat angle, weld:
Reaction is taken at the c/l of the required minimum calculated bearing length.

```
  La = .25 + Setback + .5*Min_brg
  L  = Cn_depth
  Fr = .707*PHI*Fw*Eff_weld
  K  = (9La/4L^2)^2+(1/2L)^2
 Load = SQRT[Fr^2/K]
```

(30) Plate or Tee seat weld:
(Reaction taken at .8 * the seat width from the face of the column)

```
  La = .8*Cn_width
  Fr = .707*PHI*Fw*Eff_weld
  K  = (1/2.4/Cn_depth)^2+(La/.6/Cn_depth^2)^2
 Load = SQRT[Fr^2/K]
```

(31) W Column web strength, Pl or Tee seat:
(Ellifritt & Sputo, AISC 'Engineering Journal) fourth quarter 1999, page 160)

```
  k = yield line factor
  B = seat pl length, parallel to col web
  L = stiffener length, W = stiffener width
  B' = MAX[ W/2, 2 5/8]
  e = B'/2 + .25, load eccentricity
  F_star = Fy + 2/3 * (Fu - Fy)
  m = Tw^2 * F_star / 4
  Pn = k * L * m / e
 Load  = .9 Pn
```

(32) Tee or Plate seat stiffener b/t ratio:

```
  Angle = ATN(Cn_width/Cn_depth)
  b/t   = Cn_width*COS(Angle)/Cn_thick
 Load  = Qs*PHI*1.8*Fy*Cn_width*Cn_thick
```

(33) Stiffened angle seat-to-support weld;
(Two vert. welds plus weld on heel of angle):

```
  Ew = .8(Stiff width + L thick)
  b = horiz. weld length
  d = angle vert leg dimension
  Sx = (2bd + d^2) / 3, na to heel of angle.
  Fr = .707*PHI*Fw*Eff_weld
  A = b + d + d, total weld length
  K  = (Ew/Sx)^2+(1/A)^2
 Load = SQRT[Fr^2/K]
```

```
(34) Stiffened seat angle, stiffener-to-angle weld:
      pl_w = stiff width; pl_len = stiff length
      lw = pl_w - .5;     lh = pl_len - .5,   weld lengths
      T = lw + lh
      x_bar = (lw * (.5 * lw + .5) ) / T
      y_bar = (lw * pl_len + lh * .5 * lh) / T
      ix = lh^3/12 + lh(y_bar - .5*lh)^2 + lw(pl_len - y_bar)^2
      iy = lh * x_bar^2 + lw^3 /12. + lw(.5*lw + .5 - x_bar)^2
      ip = ix + iy
      Ew = 2 * pl_w / 3     weld eccentricity
      Fr =.707*PHI*Fw*Weld_size
      R_unit = 1.0,         unit load

      point p1, bottom inside corner of stiffener
      fv = R_unit / T + R_unit * (Ew - x_bar) * x_bar / ip
      fh = R_unit * (Ew - x_bar) * y_bar / ip
      fr1 = SQRT[ fv^2 + fh^2 ]

      point p2, top outside corner of stiffener
      fv = R_unit/T + R_unit*(Ew - x_bar) * (pl_w - x_bar)/ip
      fh = R_unit*(Ew - x_bar) * (pl_len - y_bar)/ip
      fr2 = SQRT[ fv^2 + fh^2 ]

     Load = 2 * Fr / max( fr1, fr2)

(35) Stiffened seat angle, bearing on stiffener:
       An = t( b - corner_clip )
      Load = PHI * 1.8 * Fy * An

(36) Shear on support, shear connection:
      Conn. to a W or C web with a member opposite:
        Ag = Cn_depth*T_sup
      Other cases:
        Ag = 2*Cn_depth*T_sup
      (For moment connections to WF col webs, deduct
      two .75 corner clips, Cn_depth = Cn_depth  - 1.5)
      Load = PHI * .6 * Fy * Ag

(37)  Not used

(38) Single Pl shear connection yield strength:
       No axial load and dim A <= 3.5 inches:
        Fv = PHI * .6 * Fy
      Load = Fv * Ag

       Axial load or dim A > 3.5 inches:
       Refer to misc design note 32;
       T = applied tension, R = shear reaction
       sigma = T / Ag_web + R * ecc / Sx
       tau = R / Ag_web
      Load = maximum R to satisfy the yield criterion

         fv/Fv + fa/Fcr <= 1 ---
         K = 1/(Ag * Fv) + Ew/(Sx * PHI*Fcr)
         M = 1 - Compression / (Ag * PHI*Fcr)
         r2 = M/K

       Load = MIN[ r1, r2 ]

(39)  Not used
```

```
(40) Clip angle connection with axial tension,
     bolt failure mode:
      (AISC 'Engr. Journal', Vol. 22, page 65 and
       9th ed AISC, pg 4-90 )
       B = Ft*Ab          Tf = Cn_thick
       p = Cn_depth/Row   d'= Db+1/16  (Db+2 mm)
       Delta = 1-d'/p     M = p*Tf^2*Fy/4.44
       T2 = (B*a'+M)/(a'+b')
       T3 = B
       Tension = 2*Row*MIN[T2,T3]
     See design notes for 'heavy' clip angles.)

(41) Coped beams:
     (LRFD 3rd Edition, pages 9-4 thru 9-9)
      Setback = end of web to weld line or face of conn.
      e = c + Setback
      For axially loaded bms, PHI*Fcr is adjusted by + or - fa.

      Beam coped at top only:
          ho = d - dct
          c <= 2d, dct <= d/2, Cheng procedure)
          c/ho <= 1;   k = 2.2(ho/c)^1.65          (B-3)
          c/ho > 1;    k = 2.2(ho/c)
          c/d  <= 1;   f = 2(c/d)                  (B-4)
          c/d  > 1;    f = 1+(c/d)
          PHI*Fcr = 23590*f*k*(Tw/ho)^2 <= .9Fy
        Load = PHI*Fcr*Sn/e
      Beam coped at top and bottom:
           ho = d - dct - dcb
          'e' is calcuated using longer cope dim.
          c <= 2d, dct <= .2d, Cheng procedure)
          dc = top cope depth
          If web holes line up with cope: Sn = Sn_net
          fd = 3.5 - 7.5( dc / d )
            PHI*Fcr = 50840 * Tw^2 * fd/( c * ho) <= .9Fy

          Top cope depth > .2d
           Q=1, for Lambda <= .7
           Q=1.34-.486Lambda, for Lambda <= 1.41
           Q=1.3/Lambda^2, for Lambda > 1.41
           Lambda = Fy^.5 * ho /(167K*2*Tw)
           K = plate buckling coefficient, page 9-9
            PHI*Fcr = .9FyQ

        Load = PHI*Fcr *  Sn / e
      Beam coped at bottom only:
        ho = d - dcb
        c <= d2
        Load = .9Fy * Sn / e
```

```
(42) Stiffened beam web (transverse stiffener):
      b = Stiff width, t = stiff thick, l = stiff length
      Leff = Effective beam web length
             = 25*Tw, interior stiffeners
             = 12*Tw, near end of beam
      Ag = 2b*t*Pair+Tw*Leff; (Gross effective area)
      K = Tw+2b
      One pair: Ix = t*K^3/12       (in plane of web)
                Iy = Tw*Leff^3/12  ( perp to web)
      Two pairs: Stif_spa=Spacing between stiff pairs
           Ix = 2t*K^3/12
           Iy = Tw*Leff^3/12+2K*t(.5Stif_spa)^2
      Rx = SQRT[Ix/Ag];    Ry = SQRT[Iy/Ag]
      Kl/r = .75(l)/MIN[Rx,Ry]
      Aw = Tw * Leff
      Load = MIN[PHI*Fcr*Qs*Ag,
                 Aw*PHI*Fcr+PHI*1.8*Fy*Pair*2*t*(b-Fil_rad)]

(43) Clip angle connection with axial tension,
      angle failure mode:
      (AISC 'Engr. Journal', Vol. 22, No 2, page 65
       and 9th ed AISC, pg 4-90 )
       Tf = Cn_thick
       p = Cn_depth/Row   d'= Db+1/16 (Db+2 mm)
       Delta = 1-d'/p     M = p*Tf^2*Fy/4.44
       T = (1+Delta)*M/b'
      Tension = 2*Row*T
     See design notes for 'heavy' clip angles.)
```

```
(44) Moment connection to col web, flange plate weld:
       Weld_f = Weld size to col flange
       Weld_w = Weld size to col web
       Flg_l = .5(Bf_col-Tw_col)-Fil_rad-1/2; Flg weld length
       Web_l = D_col-2*Kdist_col; Web weld length
       D_prime = .95 * D_bm for welded flg pl
       D_prime = D_beam + Pl_thick for bolted flg pl
       Fr = .707 * PHI * Fw * Weld_size
      Moment = D_prime*2*(Fr*Weld_f*2*Flg_l+Fr*Weld_w*Web_l)

(45) Not used

(46) Moment connection, flange PL tension & comp:
       Agt = Ag = b*t
       Ant = t*(b-2*Dh) <= .85*Agt
       ft1 =  PHI * Fu * Ant
       ft2 =  PHI * Fy * Agt
       fc1 =  PHI * Fcr * Ag,    K = .65
      Moment = (D+t) * MIN[ft1,ft2,fc1]
      For axial load reduce moment by: Ff*(D+t)
      where Ff = the maximum flange force.

(47) Not Used

(48)  Bolt tension, four bolt extended End-Plate:
      (AISC Pgs 4-116 to 4-125)
      Interaction eqns apply to brg type bolts
      Moment = 4 * PHI * Fn * Ab * (D-Tf)

(49) Bolt shear, '4 and 8 - Bolt' extended End-Plate:
      (AISC Pgs 4-116 to 4-125)
      (Interaction equations, J3.5 & (A-J3-1) are applied
      to the tension bolts in bearing-type connections)
      Load = PHI * Fn * Ab * 2 * Row

(50) End plate thickness, 4-bolt extended End-Plate:
      (LRFD Vol II page 10-24 & Design Guide series 4)
       Bp = Eff Pl width <= Bf + 1,  Ts = End plate thick
       Af=Tf*Bf, Aw=Tw(D-2Tf), Pt=Db+1/2 (Db+12 mm)
       Wt = Fillet weld or reinforcing fillet size
       Pe = Pf-Db/4-.707*Wt     Cb=SQRT[Bf/Bs]
       Ca = (See Table 10.1)
       Am = Ca*Cb(Af/Aw)^.333(Pe/Db)^.25
       Meu = Am * Ff * Pe / 4
       Ffu = Ts^2 * PHI * Fy * Bp / (Am * Pe)
      Moment = Ffu * (D-Tf)
```

```
(51) Pl to Flange fillet weld, extended End-Plate:
      (LRFD Vol II)
        Flg_perimeter = 2(Bf+Tf)-Tw
        Fr = .707 * PHI * Fw * Weld size
      Moment = Fr*Flg_perimeter*(D-Tf)

(52) Pl to Web fillet weld, extended End-Plate:
      Minimum  weld size to develop web tension near the
      flange = PHI*Fy*Tw / (.707*PHI*Fw*2)
       Fr = .707 * PHI * .6 * Weld size
      Load = 2 * Fr * Eff_weld_len

(53) W Section col. butt plate, AISC CASE III:
      D = upper col depth     D_l=Lower col depth
      Bf = Upper col flg width,  Bf_l=Lower col flg width
       Area = Upper column area
       P = Load from upper column
       Delta = .5(T of lower col - D_u)

       Delta >= t, check shear and bend stress
       Delta <  t, check shear stress
       Po = (P/Area)(D*Tw), direct brg.
       fv = .25(P-Po)/B*t
       A = D_l-2Tf_l,  B = .5Bf_u
       Q = (Load-Po)/Bf*D
       fb = Beta*Q*B^2/t^2
       ('Formulas for Stress and Strain',
         R.J.Roark 5th ed.Table 26; 7a)
      Load = largest value of P to satisfy:
       fb <= Fb = PHI * fy & fv <= PHI * .6Fy

      Delta = 0.,  Load = Area * PHI * .6Fy

(54) Tube column butt plate:
      Upper tube size <= lower tube size)
       Ts_l Lower tube size,  Ts_u Upper tube size
       Tw_l Lower tube wall,  Tw_u Upper tube wall
       Delta = .5[(Ts_l-Tw_l)-Ts_u-Tw_u)]
       Sx = t^2/6;  per inch of plate
       Shear:   Ls = PHI*Fy*t*(Upper tube perimeter)
       Bending: Lb = PHI*Fy*(Upper tube perimeter)*Sx/Delta
      Load = MIN[Ls,Lb]

(55) Beam web tear-out, axially loaded, welded clip angle conn:
     (AISC Vol II Connections page 2-45)
      Coped beam:
      Tension = PHI*Fy*Ag,     Ag =Tw*Cn_depth

      Un-coped beam:
        Avg = (Cn_width-Clip_stbk)*Tw*2
        Atg = Cn_depth*Tw
        CaseI = PHI*.6*Fy*Avg + PHI*Fu*Atg
        CaseII = PHI*.6*Fu*Avg +.PHI*Fy*Atg
      Tension = MAX[CaseI, CaseII]
```

```
(56) Beam web tear-out, axially loaded, bolted clip angle conn:
        Sh = Horiz bolt spacing
        One bolt column-
           Anv = 2*Tw*(Lh-.5*Dh)
           Agv = 2*Tw*Lh
        More than one bolt column-
           Anv = 2*Tw*[Lh-.5*Dh+(Column-1)*(Sh-dh)]
           Agv = 2*Tw *(Column-1)Sh
        For a Coped beam: Anv = Agv = 0.

        Ant = (Spa-Dh)*(Row-1)*Tw
        Agt = Spa * (Row-1) * Tw

        CaseI = PHI(.6Fy*Agn + Fu*Ant)
        CaseII = PHI(.6Fu*Anv + Fy*Agt)
     Tension = max[CaseI, CaseII]


(57) Tension stress on axially loaded clip angle connection:
      fv/Fv + ft/Ft <= 1
      Ft = PHI*Fu,       Fv = PHI*.6Fy
      Bolted to beam:
        An = (Cn_depth - Dh * Row) * Cn_thick * 2
        Ft = PHI * Fu
      Welded to beam:
        Ag = Cn_depth * 2 * Cn_thick
        Ft = PHI * Fy
      P = Beam shear reaction
      Tension = [ 1 - P/(Area * Fv) ]*Area*Ft

(58) Combined weld stress, end plate, with axial load:
       Elastic method.
       Lw = Cn_depth - 2 * Weld_size,  (effective weld length)
       Extended end Pl: Lw = Beam T distance
       Fr = .707 * PHI * Fw * Eff_weld
       Fh = Applied axial tension / 2 / Lw
      Load = 2 * Lw * SQRT[ Fr^2 - Fh^2 ]

(59) Beam plastic moment strength:
      Moment = PHI *Fy * Zx

(60) Plate stress, end PL with axial tension load:
     (AISC 'Engr. Journal', Vol. 22, No 2, page 65
      and 9th ed AISC, pg 4-90 )
      Tf = Cn_thick   p = Cn_depth/Row   Delta = 1-d'/p
      d'= Db+1/16     M = p*Tf^2*Fy/8    T = (1+Delta)*M/b'
      Vecc = Dim from bolt group cg to beam centerline
      Tension = T/( 1/2*row + Vecc/bolt group Sx)

(61) Bolt stress, end PL with axial tension load:
      (AISC 'Engr. Journal', Vol. 22, No 2, page 65
       and 9th ed AISC, pg 4-90 )
      B = Ft*Ab          Tf = Cn_thick
      p = Cn_depth/Row   d'= Db+1/16  (Db+2 mm)
      Delta = 1-d'/p     M = p*Tf^2*Fy/8
      T2 = (B*a'+M)/(a'+ b')  with prying action
      T3 = B                  without prying action
      Vecc = Dim from bolt group cg to beam centerline
      Tension = MIN[T2,T3]/( 1/2*row + Vecc/bolt group Sx)
```

```
(62) Column bearing, AISC CASE II:
      Load = PHI*1.8*Fy(Col_brg_Area+Fill_brg_area)

(63) Coped beam web reinforced with bolted doubler PLs:
     (doubler plate stress)
       Fv = PHI*.6*Fu,     Ft = PHI*Fy
       Bm_load=Coped beam strength, see #(41)
       Tp = Doubler thickness
       Dp = Doubler depth
       Snet = Shear*Tp*Dp^2/6-Spa^2*Row*(Row^2-1)*Tp*Dh/6*Dp
       Anet = Tp*(Dp-Row*Dh)*Shear
       Shear_load = Fv*Anet
       Cope length is measured from the face of the conn.
       Moment_load = Qs*Fb*Snet/MAX[Clt+2.5,Clb+2.5]
      Load = MIN[Moment_load,Shear_load]+Bm_load

(64) Coped beam web reinforced with bolted doubler PLs:
     (doubler plate bolt shear)
      Bm_load = Coped beam strength, see #(41)
      First bolt is 2 1/2 in. (64 mm) past cope.
      Cope length is measured from the face of the connection.
      La = MAX[Clt,Clb]+2.5+(Dblr_col-1)*Col_spa*.5
        Bearing type bolts,  Rv = PHI*Rn*Shear
        Slip critical bolts, Rv = PHI*Rstr*Shear
      Load = Rv*C+Bm_load

(65) Coped beam web reinforced with bolted doubler PLs:
     (doubler bolt bearing)
      Bm_load = Coped beam strength, see #(41)
     (First bolt is 2 1/2 in. (64 mm) past cope)
       Tp = Doubler thickness
       Cope length is measured from the face of the connection.
       La = MAX[Clt,Clb]+2.5+(Dblr_col-1)*Col_spa*.5
       Rn is calculated using (J3-2a) or (J3-2c)
      Load = C*PHI*Rn * MIN[Tp*Shear,Tw] + Bm_load

(66) Coped beam web reinforced with welded doubler PLs:
     (doubler plate stress)
      Bm_load = Coped beam strength, see #(41)
       Tp = Doubler plate thickness
       Dp = Doubler plate depth
       Sx = Cn_side*Tp*Dp^2/6
       Ag = Tp*Dp*Cn_side
       Shear yielding = PHI*.6*Fy*Agv
       Cope length is measured from the face of the conn.
       Flexural yielding = .9*Fy*Sx/MAX[Clt,Clb]
      Load = MIN[Shear yielding,Flexural yielding]+Bm_load

(67) Coped beam web reinforced with welded doubler PLs:
     (doubler to web weld stress, ']' shaped weld)
       Bm_load = Coped beam strength, see #(41)
       Nws = INT[Weld_size/16], number off sixteenths in fillet weld size
       Dp = Doubler plate depth
       C = weld coefficient, table 8-9
      Load = C * Dp * Nws * Cn_side + Bm_load

(68) Bolted moment connection, flange bolt shear:
        Bearing type bolts,  Rv = PHI*Rn*Shear
        Slip critical bolts, Rv = PHI*Rstr*Ab*Shear
      (Rn for bearing type bolts is reduced by 20 percent when
       bolt pattern length > 50 in.   Table J3.2)
      Moment = Rv * 2 * Row * D
      For axial load reduce moment by: Ff*D
      where Ff = the maximum flange force.
```

```
(69) Bolted moment connection, bolt bearing on flg. conn Pl:
      Rn is calculated using (J3-2a) or (J3-2c)
      N_e = number of edge bolts
      N_i = number of interior bolts
      Moment = PHI*(Rn_e*N_e + Rn_i*N_i) (D + Cn_thick)
      For axial load reduce moment by: Ff*(D + Cn_thick)
      where Ff = the flange force.

(70) Extended end pl shear yielding:
       Bp = Eff pl width  <= Bf + 1 )
      Moment = PHI * .6 * Fy * Bp * t * 2 * (D - Tf)

(71) Wf brace bolt shear:
        Bearing type bolts,  Rv = PHI*Rn*Shear
        Slip critical bolts, Rv = PHI*Rstr*Shear
      Axial load = Rv * ( 2 * web_row + 4 * Flg_row )

(72) Bolted moment connection, bearing on beam flange:
      Rn is calculated using (J3-2a) or (J3-2c)
      N_e = number of edge bolts
      N_i = number of interior bolts
      Moment = PHI*(Rn_e*N_e + Rn_i*N_i)(.95D)
      For axial load reduce moment by: Ff*.95D
      where Ff = the flange force.

(73) Wf brace, bolt brg on brace:
      Rn is calculated using (J3-2a) or (J3-2c)
      Lc_edge = Le - .5Dh
      Lc_interior = Spacing - Dh
      N_e = number of edge bolts
      N_i = number of interior bolts

      Axial load = PHI*Rn_edge*N_e + PHI*Rn_int*N_i

(74) Wf brace tearout:
     Web connection,
       Rn1 = .6 * Fy * Agv_web + Fu * Ant_web
       Rn2 = .6 * Fu * Anv_web + Fy * Agt_web
Allow_w = PHI_J4 * max[ Rn1, Rn2]

  Flange connection,
     Rn1 = .6 * Fy * Agv_flg + Fu * Ant_flg
     Rn2 = .6 * Fu * Anv_flg + Fy * Agt_flg
Allow_f = PHI_J4 * MAX[ Rn1, Rn2]

  Tension = Allow_w + Allow_f

(75) Combined beam web stress, end PL with axial load:
      Refer to misc design note 32;
      Ae_web = Tw*(Cn_depth-2*Weld_size), fillet welds
      Ae_web = Tw*(Cn_depth-Tw),         CJP weld
      Extended end Pl: Ae_web = Tw * Bm T distance
      T = applied tension, R = shear reaction
      sigma = T / Ae_web
      tau = R / Ae_web
     Load = maximum R to satisfy the yield criterion

(76) Moment, flange-angle  connection,
     bolt failure mode:
     (AISC 'Engr. Journal', Vol. 22, No 2, page 65
      and 9th ed AISC page 4-90 )
       a = Osl-Osl_ga   b = Osl_ga-Cn_thick
       B = Ft*Ab        p = .5 conn length
       d = Db           d'= Db+1/16 (Db+2 mm)
       Tf = Cn_thick    b'= b-.5*D
       a'= a+.5*d       Delta = 1-d'/P
       M = p*Tf^2*Fy/4.44
       T2 = (B*a'+M)/(a'+b');      T3 = B
      Moment = Row*2*MIN[T2,T3]*D+2*Osl_ga)
```

```
(77) Moment flange angle conn., angle stress, tension
     and compression:
       (AISC 'Engr. Journal', Vol. 22, No 2, page 65


       a = Osl-Osl_ga      b = Osl_ga-Cn_thick
       p =.5*Cn_length
       d = Db              d'= Db+1/16 (Db+2 mm)
       f = Cn_thick        b'= b-.5*D
       a' = a+.5*d         Delta = 1-d'/p
       M = p*Tf^2*Fy/4.44  T1 = (1+Delta)*M/b'
        OSL bending --
       M1 = T1*2*(D+2*Cn_thick)
        Angle net tension --   (J5-2):
       M2 = PHI*Ant*Fu*( D + Cn_thick )
        Angle gross tension -- (J5-1):
       M3 = PHI*Agt*Fy*( D + Cn_thick )
        Angle gross compression -- (E2)
       M4 = PHI*Ag*Fcr*( D + Cn_thick )
      Moment = MIN[M1, M2, M3, M4]

(78) & (79) Not used

(80) Eight-bolt Stiffened extended End-Plate:
      Bolt stress, Shortened Desisgn Procedure
      Only A325 bolts allowed
      Interaction eqns apply to brg type bolts
      Moment = 6*Ab*Ft*(D-Tf)

(81)  Eight-bolt Stiffened extended End-Plate:
      Plate stress, Shortened Design Procedure
      Only A325 bolts allowed
       Bp = eff. end pl width <= Bf + 1.
       Pf = Db+1/2  (Db+12 mm)
       Peff = SQR[Gage^2+Pf^2]*Pf/5
      Moment = Thick^2*(D-Tf)*PHI*Fy*Bp/Peff

(82)  Not used
```

```
(83) Unstiffened column moment strength:
      (Beam flange or connection PL welded to col flange)
      Column web yield, buckling, and flange flexure:
       N = Beam flange or connection plate thickness
       dtc = dist from top of bm to top of col
       Rn1 = Fy_col*Tw_col*(N+5*K_dist_col)      (K1-2)
          if dtc <= D,
       Rn1 = Fy_col*Tw_col*(N+2.5*K_dist_col)

       Rn2 = 24*Tw_col^3*SQRT[E*Fy_col]/h        (K1-8)
          if dtc < D/2, P2 = .5P2

       Rn3 = 6.25*Tf_col^2*Fy                    (K1-1)
          if dtc < 10 * Tf_col, Rn3 = .5*Rn3

      Moment = PHI*MIN[Rn1,Rn2,Rn3]*Depth_bm*.95

(84) Not used

(85) Moment flange plate tear-out:
       Ga = Flg. gage; N = number of bolt rows
       S = Bolt spa.;  Ed = Pl end edge dist.
       b = Pl width;   t = Pl thickness
       Agv = 2*t((Row-1) * S + Ed)
       Anv = 2*t((Row-1) * S + Ed - Dh(Row-.5))
      [ shaped tearout,
        Agt = t * Ga
        Ant = t * (Ga - Dh )
        Rn1 = .6 * Fy * Agv + Fu * Ant
        Rn2 = .6 * Fu * Anv + Fy * Agt
        Case1 = PHI*MAX[Rn1,Rn2]

      L shaped tearout,
        Agt = b -  Ga
        Ant = b - (Ga - Dh )
        Rn1 = .6 * Fy * Agv + Fu * Ant
        Rn2 = .6 * Fu * Anv + Fy * Agt
       Case2 = PHI*MAX[Rn1,Rn2]

      Moment = MIN[Case1,Case1]*(Depth + t)
      For axial load reduce moment by: Ff*(Depth+t)
      where Ff = the maximum flange force.

(86) Bearing on beam web, bolted clip angle conn. with axial load:
      See design notes for 'heavy' clip angles.
       Paxial = Max. of applied ten. or comp. force
       Rn is calculated using (J3-2a) or (J3-2c)
      Load = Row*SQRT[(PHI*Rn)^2-(Paxial/Row)^2]

(87) Bearing on connection, bolted clip angle conn. with axial load:
      See design notes for 'heavy' clip angles.
       Paxial = Max. of applied tension or compression force
       Rn is calculated using (J3-2a) or (J3-2c)
      Load = Row*SQRT[(PHI*Rn)^2-(Paxial/Row)^2]
```

```
(88) Weld stress, two-side clip angle conn. with axial load:
      (Elastic method)
       Axial = Max. of applied ten. or comp. force
       A = Cn_width,  T = Axial/2,   K = A-Clip_setback
       L = Cn_depth,  W = L+2*K  (total weld length)
       X = K^2/W, c.g. location from ver weld.
       Ip = (2*K+L)^3/12 - K^2*(K+L)^2/W
       C1 = 1/W+(A-X)*(K-X)/Ip,  C2 = (A-X)*.5*L/Ip
       A1 = C1^2+C2^2,           B1 = 2*T*C2/W
       Fr = .707*PHI*Fw*Eff_weld
       C3 = (T/W)^2-Fr^2
       D1 = SQRT[B1^2-4*A1*C3]
      Load = (D1-B1)/A1

(89) Web bolt shear, combined load, clip angle conn. with axial load:
      See design notes for 'heavy' clip angles.
      Paxial = Max. of applied tension or compression force
       Bearing type bolts:
       Rv = PHI * Fn * Ab * Shear
       Slip-critical bolts at factored loads:
       Rv = PHI * Rstr  * Shear
      Load = Row*SQRT[Rv^2-(Paxial/Row)^2]

(90) Angle brace, net area tension:
       S = longitudinal bolt spacing
       n = 1 for single angle brace, 2 for double angle brace
       U = .75 two bolt rows; U = 1-x_bar/l <=.9 more than two rows
       Wg = Leg+Osl-Thickness
       Wn = Wg-Dh, for one bolt column
       Wn = Wg-Column*Dh+S^2/4/g2, two bolt columns
       Ant = Wn*t
       Ae = Ant*U
      Tension = PHI*Fu*Ae*n

(91) Brace gusset tension:
       S = longitudinal bolt spacing
       Wn = (b-Dh), for one bolt column
       Wn = (b-Column*Dh+S^2/4/g2), for two bolt columns
       Ant = MIN[Wn, .85 * b] * t
       Agt = b*t
      Tension = MIN[PHI*Fy*Agt, PHI*Fu*Ant]

(92) Angle brace tear-out, (tension loaded brace):
       t = brace thickness, S = bolt spacing
       n = 1 for single angle, 2 for double angle
       Anv = ((S-Dh)*(Row-1)+Le-.5*Dh)*t
       Agv = (S*(Row-1)+Le)*t
      One bolt column:
        Ant = (Leg-ga-.5*Dh)*t
      Two bolt columns:
        Ant = (Leg-g1 - 1.5 * Dh + S^2/4/g2)*t
      Agt = (Leg-g1)*t
       Rn1 = .6 Fy * Agv + Fu * Ant
       Rn2 = .6 Fu * Anv + Fy * Agt)
     Tension = PHI*MAX( Rn1, Rn2 )
```

```
(93) Angle brace gross area axial load:
       n = 1 for single L brace, 2 for dbl L
       Ag = (Leg+Osl-t)*t*n
      Tension or Compression = PHI*Fy*Ag

(94) Angle brace gusset plate tear-out:
      Gusset to one bm or column --
      P_edge = dim. from brace bolt line to a parallel guss. edge
      One bolt column:
        Ant = t*(P_edge - .5 * dh)
        Agt = t*P_edge
      Two bolt columns:
        Ant = t*(P_edge - 1.5 * dh + (.5S)^2/4g)


      Av = area along bolt line

      Gusset to bm & col or two bms, two bolt cols --
      Ant = t*(ga2 - dh + (.5S)^/4g)
      Agt = t*ga2
      Av = sum or area along 2 bolt lines

         Rn1 = .6 Fy * Agv + Fu * Ant
         Rn2 = .6 Fu * Anv + Fy * Agt)
      Tension = PHI*MAX[ Rn1, Rn2 ]

(95) Brace gusset Pl to beam flange fillet weld:
     (Brace gusset connected to beam and column)
       Fr = .707*PHI*Fw*Weld_size
       Min_pl = 2Fr/(PHI * .6 * fy)
       theta = included angle between beam and brace
       Pl_factor = t/Min_pl,  ( >0, <=1 )
      Axial load = Fr*2*Weld_size*Weld_len*Pl_factor/COS(theta)

(96) Wf brace gusset plate tearout:
     Under claw angles --
      Lhd = dim. between bolt cols in claw Ls
      Ant = t * ( Lhd - dh )
      Anv = t * ( S * ( Row - 1 ) + edge_dist - ( Row - .5) * dh )
      Agt = t * Lhd
      Agv = t *  S * ( Row - 1 ) + edge_dist
      Rn1 = .6Fy*Agv + Fu*Ant
      Rn2 = .6Fu*Anv + Fy*Agt
      Rto1 = PHI* MAX[ Rn1, Rn2 ]

     Under web splice --
      Ant = t * ( Col_spa - Dh )
      Anv = 2t * (S * ( Row - 1 ) + edge_dist - ( Row - .5) * Dh)
      Agt = t *  Col_spa
      Agv = 2t * (S * ( Row - 1 ) + edge_dist)
      K = ( Web_row + Flg_row )/ Web_row
      Rn1 = .6Fy*Agv + Fu*Ant
      Rn2 = .6Fu*Anv + Fy*Agt
      Rto2 = PHI* MAX[ Rn1, Rn2 ] * K

     Tension = MIN[ Rto1, Rto2 ]
```

```
(97) Brace bolt shear:
        n = number of bolts in connection
        One column; n = Row
        Two columns; n = 2*Row-1
     Bearing type bolts:
       Load = PHI * Fn * Ab * n * Shear
     Slip-critical bolts at factored loads:
       Load = PHI * Rstr * n * Shear

(98) Beam web stress, axial & shear load:
      Refer to misc design note 32;
      Ag = depth * Tw
      T = applied tension, R = shear reaction
      sigma = T / Ag
      tau = R / Ag
     Load = maximum R to satisfy the yield criterion

(99) Horiz. brace interactive gusset stress:
      Refer to misc design note 32;
      theta = included angle between brace and support
      T = applied brace tension, formula evaluated at each bm
      sigma = T * sin(theta) / Ag
      tau = T * cos(theta) / Ag
     Load = maximum T to satisfy the yield criterion

(100) Gusset to support fillet weld, no eccentricity:
      (One brace connected to gusset; gusset
      connected to one supporting member)
       Fr = PHI * Fw * .707 * Weld_size
       Min_pl = 2 * Fr / (PHI*.6 Fy)
       Pl_factor = t/Min_pl,  ( >0, <=1 )
      Axial load = Fr*2*Pl_factor*Weld_len/1.4
       ( 1.4 is the 'Richard factor')

(101) Angle brace intersection gusset compression:
       L = dim. from bolt to brace intersection point
       K = .65    (effective length factor)
       Ag = b * t
      Axial load = Fcr * Ag
```

```
(102) 'K' brace connection interactive gusset stress:
           See design note #21.
       Pp = brace force component parallel to support
       Pn = brace force component normal to support
       Eo = ecc. from C/L gusset to normal component
       Ag = Thickness*Length
       Sx = Thickness*Length^2/6
       fv = SUM[Pp]/Ag
       ft = SUM[Pn]/Ag+SUM[Pn*Eo]/Sx
       Fv = PHI * .6 Fy,   Ft = PHI * Fy
      Axial load = largest force to satisfy: fv/Fv+ft/Ft <= 1

(103) 'K' brace connection combined weld stress:
       Pp = brace force component parallel to support
       Pn = brace force component normal to support
       Fr = PHI * Fw * .707 * weld_size
       Min_pl = 2 * Fr/( PHI * Fy )
       Pl_factor = t/Min_pl,   ( >0, <=1 )
       L = gusset plate length, weld length
       Tot_weld = 2*L
       Sx = 2*L^2/6
       f1 = Pp/Tot_weld,   f2 = SUM[Pn]/Tot_weld
       f3 = SUM[Pn*Eo]/Sx
      Axial load = largest force to satisfy:
      SQRT[f1^2+(f2+f3)^2] <= Fr*Pl_factor

(104) Gusset plate stress on 'Whitmore' section:
       S2 = spacing between outside bolt cols
       Ws = Spa*(Row-1)*2*TAN(30)+S2
      Axial load = PHI*Fy*t*Ws

(105) Gusset buckling stress on 'Whitmore' section:
       Ws = Spa*(Row-1)*2*TAN(30)+g2
      Effective length factor:
         K = .5 for brace to beam & col, Vol II pg 7-113
         K = 1.2 for other cases, Vol II pg 7-138
       Kl/r = K*Lb/(t/SQRT[12]),  for a 1in. wide strip
      Axial load = PHI * Fcr * t * Ws

(106) Gusset plate interactive stress:
      (One brace connected to gusset, and gusset is
      attached to one supporting member)
       Refer to misc design note 32;
      ecc = eccentricity from the C/L of the weld
      theta = included angle between brace and support

      T = applied brace tension
      theta = included angle between brace and support
      sigma = T * sin(theta) / Ag + T * cos( theta ) * ecc / Sx
      tau = T * cos(theta) / Ag
     Load = maximum T to satisfy the yield criterion

(107) Gusset plate weld, eccentrically loaded fillets:
      (Elastic method)
       L = weld length
       Ecc = eccentricity measured from C/L of weld
       theta = angle between the weld axis and line of force
       Fr = PHI * Fw * .707 *  Eff_weld / Richard_factor
       (if the eccentricity < .5, Richard_factor = 1.4
       otherwise Richard_factor = 1.0)
       Sx = 2 * L^2 / 6
       f1 = Load * COS(theta) / 2 / L, parallel to weld axis
       f2 = Load * SIN(theta) / 2 / L, perp. to weld axis
       f3 = Load * SIN(theta) * Ecc / Sx, perp. to weld axis
      Axial load = largest value to satisfy:
      f1^2 + (f2 + f3)^2 <= Fr^2
```

```
(108) Beam web yielding and crippling (K1):
      N = length of bearing, >= K_dist
      Local Web Yielding --
        Load applied at a distance from the end of the member
        that is greater than the depth of the member,
          Rn = (N + 5*K_dist)FyTw       (K1-2)
        Load applied at or near the end of the member,
          Rn = (N + 2.5*K_dist)FyTw     (K1-3)
        Load = PHI * Rn

      Web Crippling --
          d = depth of the member
          Load applied at a distance not less than d/2 from the end
          of the member,
       Rn = 135Tw^2 * (1 + 3(N/d)*(Tw/Tf)^1.5)*SQRT[Fy*Tf/Tw]
                                      (K1-4)
          Load applied less than a distance d/2 from the end
          of the member,
       N/d <= .2,
       Rn= 68Tw^2 *(1 + 3(N/d)*(Tw/Tf)^1.5)*SQRT[Fy*Tf/Tw]
       N/d > .2
       Rn = 68Tw^2 *(1 + (4N/d- .2))*(Tw/Tf)^1.5)*SQRT[Fy*Tf/Tw]
      Load = PHI * Rn
                                      (K1-5a, -5b)

(109) Beam net section moment strength:
       Afg = Bf * Tf
       Afn = (Bf - 2 * Dh) * Tf
       T1 = .75 Fu Afn, T2 = .9 Fy Ag  (B10-1)
       If T1 >= T2:
      Moment = Fy Zx

       If T1 < T2:
        Afe = 5 Fu Afn / (6 Fy)    (B10-3)
        Deduct = Afg - Afe
        Ze = Zx - ( (1-Afe/Afg) * Afg * depth)
      Moment = Fy Ze

(110) Bearing strength at bolt holes, without eccentricity:
        N_e = number of edge bolts
        Lc = Le - .5Dh
        Rn_e = Rn      (J3-2a) or (J3-2c)

        N_i = number of interior bolts
        Lc = Spacing - Dh
        Rn_i = Rn      (J3-2a) or (J3-2c)

      Load = PHI(Rn_e * N_e + Rn_i * N_i) * Shear

(111) 'One side' clip angle weld design, L shaped weld:
       C =  weld group coefficient Table 8-44
       D = number of sixteenths of an inch in weld size
       Min_support = .707* PHI*Fw * Weld_size /(PHI*.6*Fy)
       Weld_factor = Support thickness / min_support <=1
      Load = C * D * Cn_depth * Weld_factor
```

```
(112) Shear tab weld:
      C = coefficient from Table 8-38,  k = 0
      D = number of sixteenths in weld size
      Eff fillet weld leg at HSS col wall = :
        SQRT[2] * .9 Fy * Tw / (.75 Fexx)
      Fillet weld ---
        Load = C * Cn_depth * D

      CJP weld ---
        When La <= 3.5
        Load = PHI*.6Fy * Agv
        When La > 3.5:
         fv/Fv + fb/Fb <= 1
         Fv = PHI*.6*Fy,   Fb = PHI*Fy
         K = 1 / (Ag * Fv) + Ew / (Sx * Fb)
        Load = 1/K

(113) Column moment strength, 4- or 8-tension-bolt
      extended end plate moment connection:
      (LRFD Vol II AISC Pages 10-36 to 10-39)
       Tw_c = Column web thickness
       Fy_c = Column yield strength
       Tf_c = Column flange thickness
       k = Column 'k' distance
       Depth_b = Beam depth
       Tf_b = Beam flange thickness
       Fy_b = Beam yield strength
       Pf = Distance from top of beam flange to 1st bolt
       Tp = End plate thickness
      Pb = Bolt spacing   (8-bolt end plate only)

      Col web yielding opposite bm comp flg ---
      Intermediate column locations,
       Rn = (6k + N + 2Tp) Fy Tw
      Column-end locations,
       Rn = (3k + N + 2Tp) Fy Tw
      P1 = PHI * Rn

      Column web buckling ---
      Intermediate column locations,
       Rn = 4100 Tw^3 * SQRT[Fy] / Dc
      Column-end locations,
       Rn = 4100 Tw^3 * SQRT[Fy] / (2 Dc)
      P2 = PHI * Rn

      Column flange bending at beam tension flg ---
      Four bolt,   Alpha_m = 1.36(Pe/Db)^.25
                   Bs = 2.5 (2 * Pf + Tf_b)
      Eight bolt,  Alpha_m = 1.13 *(Pe/Db)^.25
                   Bs = 2.5Pf + Tf_b + 3.5Pb
       Rn = Bs/(Alpha_m * Pe) Tf_c^2*Fy_c
      P3 = PHI * Rn
      Moment = MIN[P1,P2,P3] * (Depth_b - Tf_b)
```

```
(114) Not used

(115) Angle brace and gusset bearing check:
        N_e = number of edge bolts
        Lc = Le - .5Dh
        Rn_e = Rn     (J3-2a) or (J3-2c)

        N_i = number of interior bolts
        Lc = Spacing - Dh
        Rn_i = Rn     (J3-2a) or (J3-2c)

      Load = PHI(Rn_e * N_e+ Rn_i * N_i) * Shear

(116)'L' Shaped weld connecting a gusset to a
      column and a base or cap plate:
      (Elastic method)
      Wx = Horiz. weld length, Wy = vert. weld length
      Lw = Wx + Wy, total weld length
      theta = Angle between brace and vertical
      C_h, C_v  = Horiz. or vert. dist from the C.G. of weld group
          to the point of weld being checked
      Ecc = Perp dist. from C.G. of weld group to line of force
      Ip = polar moment inertia of the weld group
      Fr = PHI * Fw * .707 * Eff_weld * 2
      Top end of the vert weld:
        f1 = P(COS(theta)/Lw + ECc*C_h/Ip)
        f2 = P(SIN(theta)/Lw + Ecc*C_v/Ip)
       P1 = largest P to satisfy f1^2 + f2^2 <= Fr^2

      End of horiz weld farthest from wp:
        f1 = P(COS(theta)/Lw + Ecc*C_h/Ip)
        f2 = P(SIN(theta)/Lw + Ecc*C_v/Ip)
       P2 = largest P to satisfy f1^2 + f2^2 <= Fr^2
      Axial load = MIN[P1, P2]

(117 thru 119) Not used
```

```
(120) Tearout stress for plate, C web  or W web:
      Avn = (Le - .5 * Dh + (Row -1)*(Spa - Dh)) * 2 * t
      Avg = (Le + (Row -1)*Spa) * 2 * t
      Cl_spa = Spacing between center cols. for an even number of columns
      One column:             Agt = 0
                              Ant = 0
      Odd number of columns:  Ant = (Column - 1) * (Col_spa - Dh) * t
                              Agt = (Column - 1) * Col_spa * t
      Even number of columns:
        Ant = ((Column - 2)*(Col_spa - Dh)+(Cl_spa - Dh)) * t
        Agt = ((Column - 2)*Col_spa + Cl_spa)*t

       Rn1 = .6 * Fy * Agv + Fu * Ant
       Rn2 = .6 * Fu * Anv + Fy * Agt
Axial load = PHI_J4 * MAX[ Rn1, Rn2]

(121) Tee brace tearout stress
       Av = (Le - .5 * Dh + (Row -1)*(Spa - Dh)) * 2 * Tf
       At = (Bf - Flg_ga - Dh) * Tf
      Axial load = At * Ft + Av * Fv

(122) Tee or channel brace gross area axial stress:
      Axial load = PHI * Fy * Ag

(123) Tee or channel brace net section axial tension:
       Deduct = 2 * Tf * Dh        (Tee)
       Deduct = Column * Tw * Dh   (Channel)
       Ant = Agt - Deduct
       Ae = Ant * U
      Axial load = Ae * PHI * Fu

(124) Connection plate tension stress:
       Ft_n = PHI * Fu * An,   Ft_g = PHI * Fy * Ag,     (D1)
       Wg = b
       Wn = b - (Column * Dh)  <= .85 * Gross width    (B3)
       Ag = Wg * t
       An = Wn * t
      Axial load = MIN[Ft_n, Ft_g]

(125) Beam flange weld to column:
      (CJP weld, beam flange to column)
       Sw = Connecting flange plate or column flange width at weld
       M = PHI * Mp
      Moment = M * (Sw / Bf);       (Sw / Bf <= 1.0)

(126) Shear end plate, CJP weld to web:
           See design note #21.
       Fa = PHI * Fu,    Fv = PHI * .6 * Fy
       (fa/Fa + fv/Fv <=1 )
      Load = ((Cn_depth - .5) * Tw * Fy - Axial load / .6) * .4

(127) Bending and buckling on gross section of an element:
       Sx = t * d^2 / 6
      Load = PHI * Fy * Qs * Sx / La
```

```
(128) Weld stress on two 'C' shaped fillet welds,
      [], elastic method:
       l = Vertical weld segment length
       kl = Horizontal weld segment length
       al = Distance between vertical weld segments
       Weld_len = 2 * l + 4 * kl,   total length of weld
       Term1 = 1 / Weld_len + .5 * La * al / Ip
       Term2 = .5 * La * l / Ip
       Rv = PHI * Fw * .707* Weld size
      Load = SQRT[ Rv^2 / (Term1^2 + Term2^2) ]

(129) Skew fillet welds, end plate shear connection:
       Min_web = PHI Fw * Eff_throat /( PHI*.6*Fy)
       Web_factor = MIN[Tw / Min_web,1]
       Weld_len = Cn_depth - 2 * Weld_size
       Fr = PHI * Fw * Eff_throat
      Load = Fr * Weld_len * Web_factor

(130 thru 137) Not used

(138  Threaded round bar tension stress:
       F1 = PHI * .75 * Fu    Table J3.2
       F2 = PHI * Fy               (D1-1)
      Axial load = MIN[ F1, F2 ] * Agt

(139) Not used

(140) Clevis pin shear/bending stress:
         Shear:
            sl = 2 * Pin area * PHI * .6Fu
         Bending:
            c_t = clevis side pl thickness
            Sx = PI * DIA^3 / 32
            grip = t + 1/4
            bl = PHI * Fy * 4 * Sx / (grip + c_t)
      Axial load = MIN[s1, b1]
```

(141) Turnbuckle design strength, AISC Table 15-5:
(PHI = .5)

Dia D	PHI*Rn	Dia D	PHI*Rn
3/8	3.0	2	93.0
1/2	5.5	2 1/4	120.0
5/8	8.75	2 1/2	150.0
3/4	13.0	2 3/4	188.0
7/8	18.0	3	242.0
1	23.3	3 1/4	305.0
1 1/8	29.0	3 1/2	305.0
1 1/4	38.0	3 3/4	420.0
1 3/8	43.5	4	420.0
1 1/2	52.5	4 1/4	585.0
1 5/8	61.3	4 1/2	585.0
1 3/4	70.8	4 3/4	585.0
1 7/8	93.0	5	737.0

(142) Clevis design strength, AISC Table 15-3:

Clevis #	PHI*Rn (PHI = .5)
2	8.75
2 1/2	18.8
3	37.5
3 1/2	45.0
4	52.5
5	93.8
6	135.
7	171.
8	338.

(143) Not used

```
(144) Brace gusset weld to beam flange:
   (brace gusset lap welded to a beam flange)
      theta = angle between brace and beam
      Ecc = eccentricity from weld C.G. to line of brace force
      Min_t = Fw * .707 * Weld_size /(PHI*.6 Fy)
      Pl_factor = MIN[ t / Min_t, 1 ]
      Fr = PHI * Fw * .707 * Weld_size * Pl_factor
      Ecc = 0:
          Axial load =  2. * Fr * Weld_len
      Ecc > 0:
         Ix = Weld_len * SQR[.5 * Weld_spa] * 2.
         Iy = Weld_len^3 / 6.
         Ip = Ix + Iy
         k = COS( theta ) / 2. / Weld_len + Ecc * .5 * weld_spa / Ip
         m = Ecc * .5 * Weld_len / Ip + SIN( theta ) / 2. / Weld_len
Axial load = Fr / SQRT[ SQR[k] + SQR[m] ]

(145) Not used

(146) Weld stress - two parallel fillet welds,
      load not in plane of weld group:
       C = coefficient Table 8-38,  k = 0;
       D = number of sixteenths in weld size
      Load = C * Cn_depth * D

(147) Vert Brace gusset to beam weld stress:
      (See  Design Notes for interface forces)
      Elastic method
       s_weld = SQR[ weld_length] / 3
       Fr = .707 * PHI * Fw * Weld_size
       Fr = Fr / 1.4,  'Richard factor'
       min_pl = Fr /(PHI* .6 * Fy)
       when t < min_pl,   Fr = Fr * t / min_pl
       f1 = Hb / ( 2 * weld_length )
       f2 = Vb / ( 2 * weld_length ) + Mb / s_weld
       fr = SQRT[ f1^2 + f2^2 ]
      Axial load = Maximum value of P to satisfy fr <= Fr

(148) VB Gusset shear & ten. near beam flange:
      (See Design Notes for interface forces)
       Sx_pl = t * SQR[ pl_length ] / 6
       a_pl = t * pl_length
       f1 = Vb / a_pl + Mb / Sx_pl
       f2 = Hb / a_pl
       Fv = PHI* .6 Fy,  Ft = PHI * Fy
       Axial load = Maximum value of P to satisfy:
      f1 / Ft + f2 / Fv <= 1
```

```
(149) VB Beam web yielding under gusset PL:
       (See Design Notes for interface forces)
       eff_length = pl_length + 2.5 * K_dist
       Sx_web = Tw *  eff_length^2 / 6
       a_web  = Tw * eff_length
       f_allow = PHI * Fy
       f_bw = Vb / a_web + Mb / Sx_web
      Axial load = Maximum value of P to satisfy f_bw <= f_allow

(150) Vert brace Clip L bolt brg.:
      (See Design Notes for interface forces)
      Elastic method
       c = .5 * S * (Row - 1 )
       ecc = dim. from face of Ls to CG of bolt group
       Ix = Column * Row * S^2 * ( Row^2 -1)/12
       Two bolt cols --
        Iy = Row * (.5 * S)^2 * 2
       One bolt col -- Iy = 0
        Ip = Ix + Iy
       f1 = Vc / Row
       f2 = Hc / Row + ecc * Vc * c / Ip
       Rv = SQRT[ f1^2 + f2^2 ]
       Rn is calculated using (J3-2a) or (J3-2c)
      Axial load = Maximum value of P to satisfy Rv <= PHI * Rn

(151) Vert brace Gusset clip L net shear:
      (See Design Notes for interface forces)
       An = (Cn_depth - row * Dh) * (ns_t + fs_t)
      OSL bolted to column --
         Axial load = An * r * Fv / beta
      Leg bolted to gusset --
         K = ec/An + beta*g/Sx_net
      Fv=PHI*.6*Fy,  Ft=PHI * Fu

      Axial load (tension) = r/( beta/(Fv*An) + K/Ft )

(152) VB Gusset clip L OSL stress, with prying:
      (See Design Notes for interface forces)
       a = osl - osl_ga, b = osl_ga - Tf
       p = Cn_depth / Row,  d_prime = Db + .0625
       max a = 1.25 * b
       b_prime = b - .5 * Db
       a_prime = a + .5 * Db
       delta = 1 - d_prime / p
       m = p * Tf * Tf * Fy / 4.4
       t = (1 + delta) * m / b_prime
      Axial load = Max value of P to satisfy Hc <= 2 * t * Row
```

```
(153) Vert brace Gusset clip L OSL bolt shear:
      (See Design Notes for interface forces)
      Bearing type bolts:
        Vc_allow = PHI*Fn*Ab*Row*Column*Shear
      Slip-critical bolts at factored loads:
        Vc_allow = PHI*Rstr*Row*Column*Shear
      Axial load = Maximum value of P so that Vc <= Vc_allow

(154) VB Gusset clip L OSL bolt shear & ten interaction:
      (See Design Notes for interface forces)
       Fv = PHI * Fn          (Bearing bolts)
       Fv = PHI * Rstr/Ab     (SC bolts)
       Ft = PHI * Rn
       J3.5 & J3.6 interaction formulas apply
       ft = {(Hc / Row / Column) + Q} / Ab
       fv = Vc / Row / Column / Ab
      Axial load = max value of P so that ft <= Ft & fv <= Fv

(155) VB Gusset clip L to gusset bolt shear:
       (See Design Notes for interface forces)
      Elastic method
       Rv_allow = PHI * Fn * Ab  (Bearing bolts)
       Rv_allow = PHI * Rstr     (SC bolts)
       Ix = Row * Spa^2 * ( Row^2 - 1 ) / 12
       Ip = Ix + Iy
       c = .5 * Spa * ( Row - 1 )
       f1 = Vc / row
       f2 = Hc / row + Vc * Ecc * c / Ip
       Rv = SQRT[ f1^2 + f2^2 ]
      Axial load = Max value of P to satisfy Rv <= Rv_allow

(156) VB Gusset clip L to gusset weld stress:
      (See Design Notes for interface forces)
      Elastic method
       a = clip L leg,  k = horiz weld length
       l = clip L length
       w = l + k + k,       total weld length
       Ip = (2 * k + l)^3 / 12. - k^2  * (k + l)^2 / (2 * k + l)
       x = k^2 / w, loc of cg from vert weld line
       f1 = Vc / w
       f2 = Vc * ( k - x ) * ( a - x ) / Ip
       f3 = Vc * ( a - x ) * .5 * l / Ip
       f4 = Hc / w
       fr = SQRT[ ( f1 + f2 )^2 + ( f3 + f4 )^2 ]
       Fr =  PHI * Fw * .707 * Eff_weld
      Axial load = Max  value of P so that fr <= Fr

(157) VB Gusset clip L OSL bolt bearing:
      (See Design Notes for interface forces)
        P_allow_top = PHI*Rn*Column
        P_allow_lower = PHI*Rn*(Row - 1)Column
        Rn is calculated using (J3-2a) or (J3-2c)
        Pallow = P_allow_top + P_allow_lower
       Axial load = Max value of P so that Vc <= Pallow
```

```
(158) Not used

(159) Wf brace, bearing on connection:
      Rn is calculated using (J3-2a) or (J3-2c)
      Lc_edge = Le - .5Dh
      Lc_interior = Spacing - Dh
      N_e = number of edge bolts
      N_i = number of interior bolts

      Axial load = PHI*Rn_edge*N_e + PHI*Rn_int*N_i

(160) Wf brace, tension on net conn. area:
      (Web splice PLs and flg. claw Ls)
       An_web = 2 * t * MIN[ b - 2. * Dh, .85 * b]
       Web_allow = PHI * Rn * An_web

       Wg_flg = L_leg + L_osl - L_thick
       Wn_flg = Wg_flg - Dh
       An_flg =  4 * L_thick * Wn_flg
       Ae_flg = U * An_flg
       Flg_allow = PHI * Rn * Ae_flg

      Tension = Web_allow + Flg_allow

(161) Wf brace, bearing on gusset:
       Rn is calculated using (J3-2a) or (J3-2c)
       N_e = number of edge bolts
       N_i = number of interior bolts
       Web_/Flg_allow = PHI(Rn_e*N_e + Rn_i*N_i)
      Axial load = Web_allow + Flg_allow

(162) Wf brace, tension on net brace area:
      (Flange and web connected)
        Flg_deduct = Dh * Tf * 4
        Web_deduct = Dh * Tw * 2
        Ae = An = Ag - Flg_deduct - Web_deduct

      (Web only connected, shear lag)
        Web_deduct = Dh * Tw * 2
        Ae = U * ( Ag - Web_deduct )
        Pn = Fu * Ae

      Tension = PHI * Pn

(163) Not used

(164) Wf brace, tension on net conn. area:
      (Web Channel, no flange connection)
       An = Ag_c - ( Dh * Tw_c * Web_col )
       Ae = U * An * 2
       Rn = Ae * Fu
      Tension = PHI * Rn
```

```
(165)'L' Shaped weld, brace gusset to a
      column and base (or cap) Pl:
      (Uniform force method, with both
      interfaces equally stiff, Vol II pg 7-109)
      Brace W.P. at bottom of base plate.
      v = weld length to col, h = weld length to Pl
      theta = angle between brace and col
      ec  = ecc. at col, eb = ecc. at base Pl
      Alpha_bar = .5 * h,   Beta_bar = .5 * v
      K = eb * TAN(theta) - ec
      K_prime = Alpha_bar * ( TAN(theta) + Alpha_bar / beta_bar )
      D = SQR[TAN(theta)] + SQR[Alpha_bar / Beta_bar ]
      term1 = SQR[ Alpha_bar / Beta_bar ]
      Alpha = ( K_prime * TAN(theta) + K * term1 ) / D
      Beta  = ( K_prime - K * TAN(theta) ) / D
      delta_alpha = Alpha_bar - Alpha
      delta_beta  =  beta_bar - Beta
      r = SQRT[ ( Alpha + ec )^2 + ( Beta + eb )^2]
      Hb = Alpha*P/r,  Vb = eb*P/r,  Mb = Vb*delta_alpha
      Vc = Beta*P/r,   Hc = ec*P/r,  Mc = Hc*delta_beta
      top end of vert weld:
             f1 = Vc / v
             f2 = Hc / v + Mc / Sx_v
      toe end of horiz weld:
             f1 = Vb / h + Mb / Sx_h
             f2 = Hb / h
      fr = SQRT[ f1^2 + f2^2 ]

      Axial load = largest value of P so that fr <= PHI*Fw

(166) Gusset shear, horiz. brace gusset to two bms:
      Shear resistance at beam1:
        sr1 =  PHI*Rn1
      Shear resistance at beam2:
        sr2 =  PHI*Rn2
      (theta = angle between the brace and beam2)
      Axial load = MIN[ sr1/SIN(theta), sr2/COS(theta)]

(167) Gusset stress at cope, horiz. brace gusset
      connecting to two beams:
      Refer to misc design note 32;
      theta = the angle between brace and bm 1
      At beam 1:
       tau = P * cos(theta) / (b * t ),  Sx = t*b^2 / 6
       sigma = P * cos(theta) *  ecc / Sx
       P1 = max P to satisfy the yield criterion
      At beam 2:
       tau = P* sin(theta) / (b * t ),  Sx = t*b^2 / 6
       sigma = P * sin(theta) *  ecc / Sx
       P2 = max P to satisfy the yield criterion
      Axial load = MIN[ P1, P2]
```

```
(168) Bolt shear at beam connections, horizontal
      brace gusset connected to two beams:
      Bearing type bolts:
       Rv = PHI * Fn * Ab * Row * Column * Shear
     Slip-critical bolts at factored loads:
       Rv = PHI * Rstr * Row * Column * Shear
        p1 = Rv * Row1,     at beam 1
        p1 = Rv * Row2,     at beam 2
      Axial load = SQRT[ p1^2 + p2^2 ]

(169) Interactive web stress, clip L with axial load:
           See design note #21.
      Tension load ---
        Welded angle:
          Coped beam -
            Alpha = Block shear value from eqn. 7
          Uncoped beam -
            A1 = Tw * Conn_depth
            A2 = Tw * Conn_width - Setback
            Alpha = .54Fy * A1 + .75Fu * A2
           Beta = value from equation 55
        Bolted angle:
           A1 = Tw(Row - 1)(Spacing - Dh)
           Uncoped beam:
              A2 = 2*Tw(Lh - .5 * Dh)
           Beam with top or bot cope:
              A2 = Tw(Lh - .5 * Dh)
           Beam with top & bot. cope: A2 = 0.
           Alpha = PHI*.6Fu * A1 + PHI*Fu * A2
           Beta = PHI*.6Fu * A2 + PHI*Fu * A1
           t_check = Alpha * SQRT[1 - (Ten load/Beta)^2]

       Compression load ----
         Welded angle:
           A1 = Tw * Conn_depth
           A2 = Tw * Conn_width
           A2 = .5 * A2, for a coped beam
         Bolted angle:
           A1 = Tw * (Row-1) * Spacing
           A2 = Tw * Lh
           A2 = .5 * A2, for a coped beam
          Alpha = .54Fy*A1 + .75Fu*A2
          Beta = PHI*Fy * A1
          c_check = Alpha * SQRT[1 - (Comp load/Beta)^2]
       Load = MIN[ c_check, t_check ]
```

```
(170) Splice plate rupture thru net section:
        D = Cn_depth,   n = Row,   b = Spa
        Sx_net = t*D^2/6-b^2*n(n^2-1)t(Dh)/6D   AISC Pg 4-88
        La   = Setback+Lh+.5*Spa(Column-1)
        Fb = .75 * Fu
      Load = Fb*Sx_net/La

(171) Col/Bm web local yielding under brace gusset:
        Fp = PHI*Rn,     (K1-3)
        theta = included angle between brace & support
        eff_web = guss_length + 2.5 * K_dist
        eff_web_area = tw * eff_web
        Sx_web = Tw * SQR[ eff_web ] / 6.
        ecc = load eccentricity from c/l of gusset
      Axial load = Fp / ( ecc/Sx_web + 1/eff_web_area)/SIN[theta]

(172) Tension loaded end pl or clip L OSL -- gross shear:
       ( (PHI*Fy)^2 = fv_shear^2 + fv_tension^2 )
        Ag = Conn_depth * t * 2
        K = SQR[ Fv ] - SQR[ ten_load / Ag ]
       Load = SQRT[ K ] * Ag

(173) Tension loaded end pl or clip L OSL -- net shear:
       ( (PHI*Fy)^2 = fv_shear^2 + fv_tension^2 )
        An = (Conn_depth - Row * Dh) * t * 2
        K = SQR[ Fv ] - SQR[ ten_load / An ]
       Load = SQRT[ K ] * An

(174) Col. web local bending stress (axially loaded end pl or clip L):
       (Tw, Depth & K_dist are column dimensions)
       c = Distance between the top & bot bolts
       l = c + 12 * Tw;  effective web length
       For a 1 inch wide strip of web, with fixed ends:
         Sx = SQR[Tw] / 6
         L = Depth - 2*K_dist,  a = .5(L - Gage)
         P = PHI*Fy * Sx * L / a(L-a)
       Axial load = 2 * P * l

(175) Col/Bm web local bending stress, single brace gusset
      welded to web with no member framing opposite:
         d = gusset_length, e = 12 * tw, Sx_w = SQR[Tw]/4.
         L = depth - 2 * k_dist
         Rn = .9 Fy
         K = 8PHI*Rn * Sx_w (d+e)/[L(1 + 6Ecc/(d+2e)]
         theta = angle between brace and member
      Axial load = K / SIN[theta]
```

```
(176) Supporting bm web local stress (axially loaded end pl or clip L:
         a = bottom conn hole to toe of fillet
         b = dist between top & bot conn holes
         c = top conn hole to toe of filler
         L = a + b + c
         l = conn gage + 12 * tw, effective web width
         M = moment on a 1 in. wide strip of web, length L
         fixed ends, with a uniform load located over b
         w = load, kips per inch, on length b
         Sx = l * Tw^2 / 6
         Rn = PHI * Fy
         w = largest load  to satisfy M/Sx <= PHI*RN
     Axial load = w * l * b

(177) Column flange bending stress, tension
      loaded clip L or end pl connection:
      (AISC page 4-90 & 'Engr. Journal', Vol. 22, No 2, pg 65)
       p = Spacing     d'= Db+1/16 (Db+2 mm)
       Delta = 1-d'/p  M = p*Tf^2*Fy/4.44
       T = (1+Delta)*M/b'
     Tension = 2*Row*T

(178) HSS brace to gusset weld:
        (Brace notched to fit over gusset)
        HSS Connections Manual 6-18
        Ww = fillet weld size, We = Ww - 1/16
        PHI = .75,  Aw =4(.707) We * Lw
        PHI*Rn = PHI * Aw * Fw
        t1 = 1.18 * Fexx*Ww / Fy_plate
        when gusset pl thick >= t1:
      Axial load = PHI*Rn
        when gusset pl thick <  t1:
      Axial load = PHI*Rn * t/t1

(179) HSS brace net area tension:
        HSS Connections Manual 6-17
         An = Ag - 2Tw * (Guss_thick + 1/8)
         Ae = An * U
         U = 1 - x_bar / L
         Round HSS, x_bar = D / PI
         Rect. HSS, x_bar = (B^2 + 2BH)/4(B+H)
          where: H = tube dim parallel to gusset
                 B = tube dim perp to gusset
      Axial load = Ae * PHI * Fu

(180) HSS brace gross area:
      Axial load = PHI*Fy*Ag

(181) HSS brace gusset tear-out (brace welded to gusset):
      Volume II Connections, pg 2-45
         Agv = 2. * weld length * t
         Agt = brace width * t
         f1 = PHI * .6 Fy * Agv  (J5-3)
         f2 = PHI * Fu * Agt     (J5-2)
        Case1 = f1 + f2

         f1 = PHI * .6Fu * Agv   (J4-1)
         f2 = PHI * Fy * Agt     (J5-1)
        Case2 = f1 + f2
      Axial load = MAX[ Mode1, Mode2    ]
```

```
(182) HSS brace tear-out (brace welded to gusset):
        HSS Connections Manualx 6-17, shear strength
        Ae = 4 * weld_len * Tw
        Vn = .6 * Fy * Ae, PHI = .9
      Axial load = PHI * Vn2

(183) Perp. to bm. horiz. L brace gusset tearout:
      One column:
         Straight line tearout
               Rn1 = PHI*Fy*Agt)
               Rn2 = PHI*Fu*Ant)
               Mode1 = MIN[Rn1,Rn2] rn
         L shaped tearout
               Rn1 = PHI(.6Fy*Agv + Fu*Ant)
               Rn2 = PHI(.6Fu*Anv + Fy*Agt)
               Mode2 = MAX[Rn1, Rn2]
         Axial load = MIN[Mode1, Mode2]

      Two columns:
         L shaped tearout
               Rn1 = PHI(.6Fy*Agv + Fu*Ant)
               Rn2 = PHI(.6Fu*Anv + Fy*Agt)
               Mode1 = MAX[Rn1, Rn2]
         [ shaped tearout
               Rn1 = PHI(.6Fy*Agv + Fu*Ant)
               Rn2 = PHI(.6Fu*Anv + Fy*Agt)
               Mode2 = MAX[Rn1, Rn2]
         Axial load = MIN[ Mode_1, Mode_2 ]

(184) Perp. to bm. horiz. Tee brace gusset tearout:
         Straight line tearout
               Rn1 = PHI*Fy*Agt)
               Rn2 = PHI*Fu*Ant)
               Mode1 = MIN[Rn1,Rn2] rn
         L shaped tearout
               Rn1 = PHI(.6Fy*Agv + Fu*Ant)
               Rn2 = PHI(.6Fu*Anv + Fy*Agt)
               Mode2 = MAX[Rn1, Rn2]
         [ shaped tearout
               Rn1 = PHI(.6Fy*Agv + Fu*Ant)
               Rn2 = PHI(.6Fu*Anv + Fy*Agt)
               Mode3 = MAX[Rn1, Rn2]
         Axial load = MIN[Mode_1, Mode_2, Mode_3]

(185) Beam web local bending stress, single brace gusset:
      welded to web with no member framing opposite:
        d = gusset_length, e = 12 * tw, Sx_w = SQR[tw]/6
        a = dim from edge of web to c/l of gusset
        b = dim c/l of gusset to opposite edge of web
        l = a + b,       beam 'T' dimension
        theta =  angle between brace and beam
        K = 1/(d + e) * (1 + 6*ecc/( d + 2*e))
        Fb = .9 * Fy
        p1 = l^3 * Fb * Sx_w / ( 2. * sqr(a) * sqr(b)) / K
        p2 = l^2 * Fb * Sx_w / (a * b^2) / K
        p3 = l^2 * Fb * Sx_w / (b * a^2) / K
      Axial load = MIN[p1,p2,p2]/SIN[theta]
```

(186) Not used

(187) Clip L to gusset bolt shear:

```
       (See Design Notes for UFM interface forces)
       ecc = dist form heel of angle to bolt line
       f_pa = force parallel to angle's longitudinal axis
       f_pr = force perpendicular to angle's longitudinal axis
       nb = number of bolts
       Ip = polar moment of inertia of bolt group
       Brace connecting to a col & beam:
         At beam: f_pa = Hb, f_pr = Vb, Mom = Mb
         At column: f_pa = Vc, f_pr = Hc, Mom = Mc
       total_m = Mom + f_pa*ecc
       f1 = f_pa/nb + total_m * (column - 1)*col_spa * .5/Ip
       f2 = f_pr/nb + total_m * (row - 1)*spacing * .5/ Ip
       Rv = SQRT( f1^2 + f2^2 )
        (Fv from Table J3.2)
     Axial load = maximum P to satisfy Rv <= PHI*Fv*Ab
```

(188) Clip L to gusset bolt bearing:

```
       (See Design Notes for UFM interface forces)
       ecc = dist form heel of angle to bolt line
       f_pa = force parallel to angle's longitudinal axis
       f_pr = force perpendicular to angle's longitudinal axis
       nb = number of bolts
       Ip = polar moment of inertia of bolt group
       Brace connecting to a col & beam:
         At beam: f_pa = Hb, f_pr = Vb, Mom = Mb
         At column: f_pa = Vc, f_pr = Hc, Mom = Mc
       total_m = Mom + f_pa*ecc
       f1 = f_pa/nb + total_m * (column - 1)*col_spa * .5/Ip
       f2 = f_pr/nb + total_m * (row - 1)*spacing * .5/ Ip
       Rv = SQRT( f1^2 + f2^2 )
       (PHI*Rn from Spec J3.10)
     Axial load = maximum P to satisfy Rv <= PHI*Rn
```

(189 thru 192) Not used

```
(193) Wf brace net or gross area stress:
       (Both flanges bolted to gusset)
         Ag = Bf * Tf * 2 + (Depth - 2 Tf)*Tw
         An = Ag - Tf * Dh * 2 * Column
      Net area axial load = An * PHI * Fu * U
      Gross area axial load = Ag * PHI * Fy

(194) Wf brace net brace tearout:
       (Both flanges bolted to gusset)
        Ant =  2Tf * (Bf - Gage - 2 * Sec_gage - Dh )
        Agt =  2Tf * (Bf - Gage - 2 * Sec_gage )
        Anv = 4*Tf*(S(Row - 1) + End_edge - Dh(Row - .5))
        Agv = 4*Tf*(S(Row-1) + End_edge)
        Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
        Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
     Axial load = MAX[Rn1,Rn2]

(195) Guss plate tearout:
       (WF brace, both flanges bolted to gusset)
          Ant = (Gage + 2 * Sec_gage) - Dh * (Column -1)
          Ant = t * Ant * 2
          Anv = S * (Row - 1) + End_edge - Dh * (Row - .5)
          Anv = Anv * t * 4
          Agv = t * 4 *(Spacing*(Row-1) + End_edge)
          Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
          Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
        Axial load = MAX[Rn1,Rn2]

(196) Intersecting gusset net/gross area:
       (WF brace, web horiz. both flgs bolted to gusset)
          Ag = 2 * Width * Thick
          An = 2(Width - Column * Dh) * Thick <= .85Ag
      Net area axial load   = PHI * An * Fu
      Gross area axial load = PHI * Ag * Fy

(197) Gusset clip L stress, leg to gusset:
       (WF brace, both flgs bolted to gusset)
         theta = angle between brace and member
         Sx = Angle_thick * SQR[Angle_length]/6
         Area = Angle_length * Angle_thick
         Fv = PHI * Fy,    Ft = PHI * .6Fy
         (fv/Fv + ft/Ft <= 1)
         K = SIN(theta) / Area + Ecc * SIN(theta)/Sx
         P = 1. / ( COS(Phi)/(Area * Fv) + K / Ft)
      Axial load = 2P

(198) Gusset clip L stress, leg to bm/col flange:
       (WF brace, both flgs bolted to gusset)
          theta = angle between brace and member
          Oh = dim from toe of sprtg. flng to heel of angle
          A = Angle_length * Angle_thick
          Sx = Angle_thick * SQR[Angle_length] / 6
          Fb = Fy,  Fv = PHI* .6Fy
          T1 = Sx * Fb / ( SIN[theta] * Oh )
          K = SQR[SIN(theta) / A] + SQR[COS(theta) / A])
          T2 = SQRT[ SQR[ Fv] / K ]
      Axial load = 2 * MIN[T1, T2]
```

```
(199) Gusset to clip L weld stress:
      (WF brace, both flgs bolted to gusset)
         theta = angle between brace and member
         Tl = 2. * angle_length
         Fr = PHI * Fw * .707 * Weld_size
         k = SQR[ COS[theta] / Tl] + SQR[SIN[theta] / Tl]
         p = SQRT[ SQR[Fr] / k ]
       Axial load = 2 p

(200) WF brace, web horiz, weld stress, angle
      connection to supporting member:
        theta = angle between brace and support
         Fr = PHI * Fw * .707 * Weld size
         L = weld length,  Sx = 2 * L^2 / 6
         K = ecc * SIN(theta) / Sx + SIN(theta) / 2L
       Axial load = 2SQRT[Fr^2 /(COS(Theta)/2L)^2 + K^2]

(201) Shear tee shop bolts, with eccentricity:
        J3.7 & J3.9b interaction formulas apply
        Applied tension = P * Ecc / Sx of bolt group
        Ecc = Dist. from flg of tee to C.G. of bolt group in stem
        ft = (Applied tension + Q)  / Ab
        fv = P / (Row*Column*Ab)
        Fv = PHI*Fn;  (PHI*Rstr/Ab, for SC bolts)
      Load = max value of P for ft <= PHI*Ft & fv <= Fv

(202) Bolt bearing, skew End PL, axial load:
       theta = angle between beam axis and perp. to conn.
       F = MAX[ Axial ten., Axial compr. ]
       K = SQRT [SQR[PHI*Rn_edge] - SQR[ F*SIN[theta] ]]
       M = SQRT [SQR[PHI*Rn_int] - SQR[ F*SIN[theta] ]]
       Rn_ is calculated using (J3-2a) or (J3-2c)

      Load = (K + M * (Row-1)) * Column

(203) Net PL stress, axially loaded shr tab:
           See design note #21.
        fv/Fv + ft/Ft <= 1
        Fv = PHI * .6Fy,     Ft = PHI * Fu
        k = 1. / (Fv * An )  + Eb / ( Ft * Sx_net)
      Load = ( 1. - Axial load / (Ft * An) ) / k

(204) Bolt shear, shr tab with axial load:
        Nb = number of bolts
        c  = .5 * Spacing * (Row - 1)
        P  = vertical reaction
        r1 = Axial load / Nb + P * eb * c / Ip bolt group
        r2 = P / Nb
        Rv_allow = PHI*Fn*Ab;  (PHI*Rstr, for SC bolts)
      Load = largest P to satisfy:
             SQR[r1] + SQR[r2] <= Rv_allow

(205) Bolt bearing, shr tab with axial load:
         Nb = number of bolts
         c  = .5 * Spacing * (Row - 1)
         P  = vertical reaction
         r1 = Axial load / Nb + P * eb * c / Ip bolt group
         r2 = P / Nb
         Rn is calculated using (J3-2a) or (J3-2c)
      Load = largest P to satisfy:
             SQR[r1] + SQR[r2] <= PHI*Rn
```

```
(206) Net area tension, shr tab with axial load:
         d = Cn_depth - Row * Dh <= .85 * Cn_depth
         Anet = t * d
         Block shear tearout:
          Rn1 = PHI(.6*Fy*Agv + Fu*Ant)
          Rn2 = PHI(.6*Fu*Anv + Fy*Agt
          Rbs =  MAX( Rn1, Rn2)
      Axial load = MIN[ PHI* Fu * Anet, Rbs ]

(207) W Column splice, flange Pl bolt shear:
         N = number of bolts on one side of splice
         Rv_allow = PHI*Fn*Ab;  (PHI*Rstr, for SC bolts)
      If input uplift > 0,
        Uplift = 2(Rv_allow * N - Mom_col/D_col)
      If input uplift = 0,
        Moment = Rv_allow * N * D_col

(208) W Column splice, flange Pl net/gross area:
         net_w = Pl_w - 2 * dh <= .85 * Pl_w
         An = t * net_w,  Ag = t * Pl_w
         F = MIN[ PHI*Ag*Fy, PHI*An*Fu, PHI*Ag*Fcr ]
         La = D_column + t
      If input uplift > 0,
       Uplift = 2( F - Mom_col / La )
      If input uplift = 0,
       Moment = F * La

(209) W Column splice, bolt brg on flange Pl:
       La = D_column + t
       N_e = number of edge bolts, one side of splice
       N_i = number of interior bolts, one side of splice
       Rn is calculated using (J3-2a) or (J3-2c)
      If input uplift > 0,
       Uplift = 2(( Rn_e*N_e + Rn_i*N_i) - Mom_col/La)
      If input uplift = 0,
       Moment = PHI * (Rn_e*N_e + Rn_i*N_i) * La

(210) W Column splice bolt brg on flange:
       N = number of bolts on one side of splice
       La = D_column - Tf
       Rn is calculated using (J3-2a) or (J3-2c)
      If input uplift > 0,
       Uplift = 2(( Rn_e*N_e + Rn_i*N_i) - Mom_col/La)
      If input uplift = 0,
       Moment = PHI * (Rn_e*N_e + Rn_i*N_i) * La
```

```
(211) W net section allowable moment with axial tension:
      Afg = Bf * Tf
      Afn = (Bf - 2 * Dh) * Tf

    If .75Fu*Afn >= .9Fy*Afg        (B10-1)
     Moment = (PHI*Mn - ft) * Zx

    If .75Fu*Afn < .9Fy*Afg         (B10-2)
       Afe = 5 * Fu * Afn / (6 Fy)  (B10-3)
       Pct_red = 1 - Afe / Afg
       Zx_e = Zx-2(Pct_red * Afg * Depth/2)
     Moment = (PHI*Mn - ft) * Zx_e

(212) Beam moment connection PL to column flange weld:
       Fillet weld --
 Eff_weld = PHI*Fy * t/(.707 * 2 * PHI *Fw)
                  <= actual fillet size
         f = .707 * Eff_weld * PHI * Fw * 2 * Weld_len
       CJP weld --
         Agt = t * Weld_len
         f = Agt * PHI * Fw
      Moment = f * (D + t)

(213) Extended clip L, supported mbr web bolt shear:
          Eccentricity considered --- Elastic method
          AISC Manual, 8th ed, page 4-58
          n = Row - 1, bolt is skipped on 1 row
          C = vert dist from bolt group n/a to extreme bolt
          t1 = SQR[ La_x * C / Ip ]
          t2 = (1/(Column*n) + La*Col_spa/(2*Ip))^2
       Bearing type bolts:
         Rn = PHI * Fn * Ab * Shear
       Slip-critical bolts at factored loads:
         Rn = PHI * Rstr * Shear
       Load = SQRT[ SQR[PHI*Rn]/(t1 + t2)]

(214) Extended clip L, supported mbr web bolt bearing:
       Eccentricity considered --- Elastic
       method AISC Manual, 8th ed, page 4-58
          n = Row - 1, bolt is skipped on 1 row
          C = vert dist from bolt group n/a to extreme bolt
          t1 = SQR[ La_x * C / Ip ]
          t2 = 1/(Column*n) + La*Col_spa/(2*Ip)
          t2 = SQR[ t2 ]
       Load = SQRT[ SQR[PHI*Rn*Shear]/(t1 + t2)]

(215) HSS brace gusset Whitmore area yield stress:
         delta = Weld_len * TAN(30)
         Ws = H + 2*delta
      Axial Load = PHI * Fy * t * Ws
```

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(216) HSS brace gusset plate buckling stress:
         Brace field welded to gusset:
           delta = Weld_len * TAN(30)
           Ws = H + 2*delta
           Fa from E2-1 or E2-1, with eff. length factor ---
           K = .5 for gusset connecting to col and beam
           K = 1.2 for all other gussets
           Ag = Ws * t
      Axial Load = PHI * Ag * Fcr

(217) HSS brace gusset gross area ten. stress:
         delta = Weld_len * TAN(30)
         Ws = H + 2*delta
         Ag = t * MIN[Ws, b]
      Axial load = PHI * Ag * Fy

(218) HSS brace intersection gusset gross area compr. stress:
         delta = Weld_len * TAN(30)
         Ws = H + 2*delta
         Ew = MIN[Ws, b], effective width
         Ag = Ew * t
         K = 1.2 (effective length factor)
      Axial load = PHI * Fcr * Ag

(219) Starred L brace-to-gusset bolt shear:
        Two cols, Number of bolts  = 2 * Row
        4 cols,   Number of bolts  = 4 * Row - 2
     Bearing type bolts:
       Axial load = PHI * Fn * Ab * Number of bolts
     Slip-critical bolts at factored loads:
       Axial load = PHI * Rstr * Number of bolts

(220) Starred L brace gusset tearout:
        b_b = Back to back spacing of angles in the plane
              of the gusset.
        Two bolt cols -
           Ant = t(b_b + 2*g1 - Dh)
           Anv = 2t[(Row - 1)*(S - Dh) + Edge_dist - .5Dh]
           Agt = t(b_b + 2*g1)
           Agv = 2t[(Row - 1)*S + Edge_dist

        Four bolt cols -
           Ant = t(b_b + 2(g1+g2) - 3Dh + 2*S_g)
           Anv = 2t[(S-Dh)(Row-2) + .5S + Edge_dist -.5Dh]
           Agt = t(b_b + 2(g1+g2)
           Agv = 2t[S(Row-2) + .5S + Edge_dist
           Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
           Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
       Axial load =  max( Rn1, Rn2)

(221) Starred L brace gusset PL stress:
         Agt = t * b
           Net_width = t - Column * Dh + add  <=  .85 b
           Two cols, add = 0.;       4 cols, add = 2 * S_g
         Ant = t * Net_width
      Axial load = MIN[PHI*Fy*Agt, PHI*Fu*Ant ]

(222) Vert brace gusset clip angle leg gross stress:
      (See Design Notes for interface forces)
          ft/Ft + fv/Fv <= 1
          Ft = PHI * Fy,   Fv = PHI * .6Fy
          k = ec / Ft + beta / Fv
      Axial load =  r * Ag / k
```

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(223) Not used

(224) Wide flange vert. brace claw angle block shear tearout:
          Ant = 4*t(Angle leg - g - .5 * Dh)
               (At checked for each angle leg)
          Anv = 4*t(Spa * (Row - 1 ) + Le - (Row - .5) * Dh)
          Agt = 4*t(Angle leg - g
               (At checked for each angle leg)
          Agv = 4*t(Spa * (Row - 1 ) + Le

          Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
          Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
          Rbs =  MAX[ Rn1, Rn2]
      Axial load = Rbs * A / Af

(225) Wide flange vert. brace web lap plate tearout:
          Ant = 2 * t MIN[ Col_spa - Dh, Wg - Col_spa - Dh ]
          Anv = 4 * t(Spa(Row - 1 ) + Le - (Row - .5)Dh)
          Agt = 2 * t MIN[ Col_spa, Wg - Col_spa ]
          Agv = 4 * t(Spa(Row - 1 ) + Le )
           Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
           Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
           Rbs =  MAX[ Rn1, Rn2 ]
      Axial load = Rbs * A / Aw

(226) Wide flange vert. brace web channel tearout:
          Ant = (Col_spa - Dh) * Tw * 2
          Anv = 4*Tw(Spa * (Row-1) + Le - (Row - .5) * Dh )
          Ang = Col_spa * Tw * 2
          Anv = 4*Tw(Spa * (Row-1) + Le )
          Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
          Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
      Axial load =  MAX[ Rn1, Rn2 ]

(227) HSS brace, welded Pl Tee ftg., tee stem to cap Pl weld:
          Fw = PHI * .6Fexx(1 + .5 SIN(90)^1.5)
          Eff throat = PHI*.6 * Fy * t / (2 * Fw)
                      <= .707 * Weld leg size
          Fr = Eff throat * PHI * Fw
      Axial load = Fr * 2. * Stem Pl width

(228) HSS brace, welded Pl Tee ftg., tee stem tearout:
          Ant = t(Col_spa - Dh)
          Anv = 2t(Le + S*(Row - 1) - (Row - .5)Dh)
          Agt = t(Col_spa)
          Anv = 2t(Le + S*(Row - 1))
          Rn1 = PHI(.6 * Fy * Agv + Fu * Ant)
          Rn2 = PHI(.6 * Fu * Anv + Fy * Agt)
      Axial load =  MAX[ Rn1, Rn2 ]

(229) HSS brace, welded Pl Tee ftg., cap PL to tube weld:
          Fw = PHI*.6Fexx(1 + .5 SIN(90)^1.5)
          L = 2(5Tc + Ts + 2Ws)
      Axial load =  Eff_throat * Fw * L
```

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(230) HSS brace, welded Pl Tee ftg., guss. or tee stem buckling:
          Fcr from equations (E2-1,-2,-3) with:
             K = 1.2 for tee stem plate
             K = .5 for gusset connecting to col and beam
             K = 1.2 for all other gussets
          Ecc = (T_gusset + Ts) /2
          Ew = Stem pl width; (stem pl width + 1 for guss Pl)
          Zx = Ew * SQR[t] / 4
          PHI*Mn = PHI * Fy * Zx
          PHI*Pn = PHI * Fcr * Ag
          Mu = Pu * .5 * Ecc
      Axial load = Largest value of Pu to satisify
                   AISC equations (H1-1a) & (H1-1b)

(231) HSS brace, welded Pl Tee ftg., cap plate shear:
         PHI = .75
         U = Ts + 2Ws
         Pn = 2(Ws*Tc * .6 * Fy + Tw_hss * U * Fy_hss
      Axial load = PHI * Pn

(232) HSS brace, welded Pl tee ftg., tube wall strength:
         Ws = stem to cap pl weld leg size
         Ae = 2Tw(5*Tc + Ts + 2Ws)
      Axial load = PHI*Fy * Ae

(233) VB Gusset - column flange stress, with prying:
      (See Design Notes for interface forces)
       a = .5*(Col flg ga - tw), b = clip gage - clip thick
       p = Cn_depth / Row,  d_prime = Db + .0625
       max a = 1.25 * b
       b_prime = b - .5 * Db
       a_prime = a + .5 * Db
       delta = 1 - d_prime / p
       m = p * Tf * Tf * Fy / 4.44
       t = (1 + delta) * m / b_prime
      Axial load = Max value of P to satisfy Hc <= 2*t*Row

(234) Beam splice with axial load, web bolt shear:
        n = number of web bolts
        t = web portion of axial load
        p = vertical load at splice
        f1 = p/n + p*Ecc(Column-1)Col_spa * .5 / Ip
        f2 = T/n + p*Ecc*(Row-1)*S * .5 / Ip
        SC bolts:
          Rv = PHI*Rstr*Ab * shear
        Brg bolts:
          Rv = PHI*Fn*Ab * shear
      Load = max p for SQRT[f1^2 + f2^2] <= Rv

(235) Beam splice with axial load, web bolt bearing:
        n = number of web bolts
        T = web portion of axial load
        p = vertical load at splice
        f1 = p/n + p*Ecc(Column-1)Col_spa *.5 / Ip
        f2 = T/n + p*Ecc*(Row-1)*S *.5 / Ip
        Rn is calculated using (J3-2a) or (J3-2c)
      Load = max p for SQRT[f1^2 + f2^2] <= PHI*Rn
```

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(236) Beam splice with axial load, web plate gross area stress:
       Shear, axial interaction: fv/Fv + fa/Fa <= 1
       T = web portion of tension force
       C = web portion of compression force
       Fv=PHI*.6*Fy, Ft=PHI*Fy, Fc=PHI_c*Fcr
       K=.65, l=dist between inside bolt cols.
                for Fcr calculation.
       Tension load check --
         tmp = 1/(Ag*Fv) + Ecc/(Sx*Ft)
         Ru_t = (1 - T/(Ag*Ft) ) / tmp
       Compression load check--
         tmp = 1/(Ag*Fv) + Ecc/(Sx*Fc)
         Ru_c = (1 - C/(Ag*Fc) ) / tmp
       Load = MIN[Ru_t, Ru_c]

(237) Beam splice with axial load, web plate net area stress:
       Shear, axial interaction: fv/Fv + fa/Fa <= 1
       T = web portion of tension force
       C = web portion of compression force
       Fv=PHI*.6*Fy, Ft=PHI*Fu, Fc=PHI_c*Fcr
       K=.65, l=dist between inside bolt cols.
                for Fcr calculation.
       Tension load check --
         tmp = 1/(An*Fv) + Ecc/(Sx_net*Ft)
         Ru_t = (1 - T/(An*Ft) ) / tmp
       Compression load check--
         tmp = 1/(An*Fv) + Ecc/(Sx_net*Fc)
         Ru_c = (1 - C/(An*Fc) ) / tmp
       Load = MIN[Ru_t, Ru_c]

(238) Through plate weld to HSS column:
        H = HSS dim parallel to thru plate
        Ww = fillet weld leg size
        Aw = .707 * Ww * weld length * 2
        Ru = .75 * .6 * Fexx * Aw
        t1 = .58917 * Fexx * Ww / Fy_col
        For t1 > Tw_col, Ru = Ru * Tw_col/t1
      Load = Ru( H + Ew ) / H

(239) HSS wall compression yielding under cap pl:
       (HSS Connections Manual formula (7-2) )
       Rn = (5 * t1 + N) * Fy * t <= B*Fy*t
       N = 2*k dist for W beams, 2 in. for joists
       t = HSS wall thickness, t1 = cap pl thick
       B = HSS dimension perp to beam
       PHI = 1.0
      Load = PHI * Rn
```

```
(240) HSS wall compression crippling under cap pl:
       (HSS Connections Manual formula (7-3) )
       Rn = .8 * t^2 (1 + 3(N/.5B)(t/t1)^1.5 )sqrt(E*Fy (t1/t))
       t = HSS wall thickness, t1 = cap pl thickness
       N = 2*k dist for W beams, 2 in. for joists
       B = HSS dimension perp to beam
       Fy = yield strength of HSS
       E = modulus of elasticity of the HSS
       PHI = .75
      Load = PHI * Rn

(241) HSS cap PL flexural strength, compressive reaction:
       (HSS Connections Manual formula (7-1) )
       Rn = (B * t1^2 / 4(Nr/2 + a - H/2))Fy_pl
       t1 = cap pl thickness
       Nr = bearing length of attached member
       a = dist from HSS centroid to end of beam
       B = HSS dimension perp to beam
       H = HSS dimension parallel to beam
       PHI = .90
      Load = PHI * Rn

(242) W brace, web horiz, support flg local bending stress:
       theta = acute angle between brace and support member
       Ecc = .5 * Brace depth - support mbr k1 dist.
       Eff flg length = guss length + 2Ecc*Tan(30)
       Sx = Eff flange length * Tf^2 / 6
       Mr = Flg_Fy * Sx
       Axial load = Mr * 2. / (Ecc *  SIN(theta) )

(243) W brace, web horiz, to bm & col, beam flg local bending:
        (See Design Notes for interface forces)
        Ecc = .5 * Brace depth - bm k1 dist.
        Eff flg length = guss length + Ecc*Tan(30)
        Sx = Eff flg length * Tf_bm^2 / 6
        Mr = Flg_Fy * Sx
      Axial load = Max P so that
                   .5 * Vb * Ecc <= Mr

(244) W brace, web horiz, to bm & col, column flg local bending:
       (See Design Notes for interface forces)
       Ecc = .5 * Brace depth - col k1 dist.
       Eff flg length = guss width + 2*Ecc*Tan(30)
       Sx = Eff flg length * Tf_col^2 / 6
       Mr = Flg_Fy * Sx
      Axial load = Max P so that
                   .5 * Hc * Ecc <= Mr

(245) W brace, web horz, to bm & col, guss gross stress at bm:
       (See Design Notes for interface forces)
        Eff guss length = conn angle length
        ft = Vb / Agt + Mb / Sx
        fv = Hb / Agv
      Axial load = Max P so that ft/PHI*Rn + fv/PHI*Rn <=1
```

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(246) W brace, web horz, to bm & col, guss gross stress at col:
       (See Design Notes for interface forces)
        Eff guss length = conn angle/bar length
        ft = Hc / Agt + Mc / Sx
        fv = Vc / Agv
      Axial load = Max P so that ft/PHI*Rn + fv/PHI*Rn <=1

(247) W brace, web horz, to bm & col, guss net stress at bm:
       (See Design Notes for interface forces)
        Eff guss length = conn angle length
        ft = Vb / Ant + Mb / Sx_net
        fv = Hb / Anv
      Axial load = Max P so that ft/PHI*Rn + fv/PHI*Rn <=1

(248) W brace, web horz, to bm & col, guss net stress at col:
       (See Design Notes for interface forces)
        Eff guss length = conn angle/bar length
        ft = Hc / Ant + Mc / Sx_net
        fv = Vc / Anv
      Axial load = Max P so that ft/PHI*Rn + fv/PHI*Rn <=1

(249) W brace, web horz, to bm & col, guss L OSL stress:
         (See Design Notes for interface forces)
         T_allow from angle failure mode including prying
         AISC manual page 11-11,analysis for prying action.
         N = number of bolts in connection
      Clip angles at column --
         T_applied = Hc / N + tension from moment
      Clip angles at beam --
         T_applied = Vb / N + tension from moment

(250) W brace, web horz, to bm & col, col/bm flg bending:
         (See Design Notes for interface forces)
         T_allow from flange failure mode including prying
         AISC manual page 11-11,analysis for prying action.
         N = number of bolts in connection
      Column flange --
         T_applied = Hc / N + tension from moment
      Beam flange --
         T_applied = Vb / N + tension from moment
      Axial load = Max P so that T_applied <= T_allow

(251) W brace, web horz, to bm & col, clip L bolts to beam
      or column  -- combined tension and shear:
         (See Design Notes for interface forces)
         N = number of bolts in connection
         T_m = bolt tension from moment, Mb or Mc
         At col: ft = (Hc / N + T_m + Q) / Ab
                 fv = (Vc / N * Ab )
         At bm:  ft = (Vb / N + T_m + Q) / Ab
                 fv = (Hb / N * Ab )
      Axial load = Max P to that ft <= Ft from
                   J3-5 and J3-6

(252) Shear end PL connection block shear:
       Rn_a = .6Fy*Agv + Fu*Ant     (J4-3a)
       Rn_b = .6Fu*Anv + Fy*Agt     (J4-3b)
      Load = PHI* MAX[ Rn_a, Rn_b ]
```

```
(253) Clip L connection block shear:
        Rn_a = .6Fy*Agv + Fu*Ant      (J4-3a)
        Rn_b = .6Fu*Anv + Fy*Agt      (J4-3b)
        K = PHI* MAX[ Rn_a, Rn_b ]
        Rbs_ns = min value of K from Web leg & Osl
        Rbs_fs = min value of K from Web leg & Osl
      Load = Rbs_ns + Rbs_fs

(254) HSS brace, erection pin bending:
       (Principal stress method)
       Pin material Fy = 36 ksi
       P = axial force in brace
       L = ctr to ctr of HSS wall
       fv = .5 * P / Ab
       fb = P * L / (4 * Sx)
       fv_max = SQRT[ .25 * fb^2 + fv^2 ]
       fb_max = fb * .5 + SQRT[ .25 * fb^2 + fv^2 ]
     Load = max value of P to satisfy:
       fv_max <= PHI * .6 * Fy
       fb_max <= Fy

(255) HSS brace, erection pin bearing:
        Rn1 = PHI*1.8 * Fy_hss * Bdia * Tw * 2
        Rn2 = PHI*1.8 * Fy_pl * Bdia * t_pl
      Load = MIN[ Rn1, Rn2 ]

(256) HSS column wall yield line, at stiffened
      plate or tee seat:
        HSS design manual formula 4-21
        L = stiff. depth,  W = Stiff. width
        B = HSS width
        PHI = .9,         e = .8 * W
        f = 1/(B - .2L),  g = (1 + .661B/L)
        h = SQRT[ (B-.4L)(7B + .4L)]
        m = B(B -.4L)/(4L),  n = 2L + 2.56B
        k = f( gh + m + n )
        Rn = k * t^2 * Fy_col * L / (4e)
      Load = PHI * Rn

(257) & (258) Not used
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(259) Beam bolted moment conn., flange tearout:
        Ga = Flg. gage; N = number of bolt rows
        S = Bolt spa.;  Ed = flg end edge dist.
        Agv = 2*Tf((Row-1) * S + Ed)
        Anv = 2*Tf((Row-1) * S + Ed - Dh(Row-.5))

      L shaped tearout,
        Agt = Bf -  Ga
        Ant = Bf - (Ga - Dh )
        Rn1 = .6 * Fy * Agv + Fu * Ant
        Rn2 = .6 * Fu * Anv + Fy * Agt
        Rbs = PHI*MAX[Rn1,Rn2]

      Moment = Rbs*(Depth - Tf)
      For axial load reduce moment by: Ff*(Depth-Tf)
      where Ff = the maximum flange force.

(260) Col/Bm local web crippling under vert. brace gusset:
        N = gusset length,  ecc = ecentricity from gusset c/l
        Theta = angle between the brace and support
        A_web = N * Tw,       web area
        Sx_web = Tw * N * N / 6
        Formulas (K1-4) & (K1-5a, -5b):
      Load applied at a dist. >= d/2 from top of column,
        Rn =135Tw^2[1 + 3(N/d) (Tw/Tf)^1.5]SQRT[Fy*Tf/TS]

      Load applied at a dist. <  d/2 from top of column,
        N/d <= .2
        Rn =68Tw^2[1 + 3(N/d) (Tw/Tf)^1.5]SQRT[Fy*Tf/TS]

        N/d > .2
        Rn =68Tw^2[1 + (4N/d - .2) (Tw/Tf)^1.5]SQRT[Fy*Tf/TS]

        K = 1 / A_web + ecc / Sx_web
        Rn' = ( Rn / A_web * 1 / K )
       Axial load = Rn' / SIN(theta)

(261) Beam local web crippling under vert. brace gusset:
       (Brace gusset connecting to a col and beam. See
         design notes for interface forces)
        N = gusset length
        A_web = Tw * N,    Sx_web = Tw * N * N / 6
        For N/d <= .2
          Rn = 68Tw^2 [1+3(N/D)(Tw/Tf)^1.5] * SQRT[Fy*Tf/Tw]
                                              (K1-5a)
        For N/d > .2
          Rn = 68Tw^2 [1+(4N/D - .2))(Tw/Tf)^1.5] * SQRT[Fy*Tf/Tw]
                                              (K1-5b)
        Fp = PHI *  Rn / A_web
        fp = Vb / A_web + Mb / Sx_web
      Axial load = maximum P to satisfy fp <= Fp
```

```
(262) Concentrated longitudinal load on HSS face:
       HSS Connections Manual, formula  8.2-2
       t = col wall design thickness
       t1 = conn plate thickness
       N = loaded plate length, parallel
           to the column axis
       B = column face width
       PHI = 1.0,   Qf = 1.0
       term1 = Fy * t^2 / ( 1 - t1/B)
       theta = angle between col and applied load
       term2 = 2N/B + SQRT[ 1 - t1/B]
       Rn = term1 * term2 * Qf

      Axial load = PHI * Rn / SIN(theta)

(263) HSS gross shear:
       HSS Connections Manual, Specification 5.2
       H = overall height
       t = design wall thickness
       h = H - 3t
       Aw = 2 * H * t    (5.2-4)
       Vn = Fn * Aw      (5.2-3)

       (i) h/t <= 2.45 SQRT[E/Fy]
          Fn = .6Fy      (5.2-5)

       (ii) h/t <= 3.07 SQRT[E/Fy]
          Fn = .6Fy( 2.45 * SQRT[E/Fy]/(h/t), (5.2-6)

       (iii) h/t <= 260
          Fn = .458 * PI^2 * E/SQR[h/t], (5.2-7)

      Load = PHI * Vn

(264) Concentrated transverse force on an HSS face:
       HSS Connections Manual, Specification 8.1
       See HSS connections manual for determination
       of connection resistance reduction factor Qf.
        SDS2 design basis, Qf = 1
        b1 = width of the loaded plate
        t1 or N = thickness of the loaded plate
        t = HSS design wall thickness
        B = overall HSS width
        h = flat side of HSS wall, B - 3t
        k = HSS outside corner radius, 1.5 * t
       Round HSS
        PHI = 1
        D = HSS diameter
        Rn = Qf * 5Fyt^2/(1 - .81*b1/D) (8.1-1)
```

```
(264) Continued
       Rectangular HSS

        PHI = 1.0
        Rn = 10Fy* t*b1 / (B/t)         (8.1-2)

        Web yielding, PHI = 1.
        Rn = 2Fy*t*(5k + N)            (8.1-3)
           = 2Fy*t*(2.5k + N);   at end of HSS

        Web crippling, PHI = .75
        Rn = 1.6 * t^2[1+3N/h]SQRT[E/Fy]  (8.1-4)

        When b1 > .85B and < B - 2t:
          PHI = 1.0
          bep = 10*b1/(B/t) <= b1
          Rn = .6Fy * t(2*t1 + 2*bep)     (8.1-5)

      Load = PHI * MIN[ Rn ]

(265) Concentrated longitudinal force on an HSS face:
       HSS Connections Manual, Specification 8.2
       See HSS connections manual for determination
       of connection resistance reduction factor Qf.
        SDS2 design basis, Qf = 1,

        t1 = loaded plate thickness
        N = length of the loaded plate
        t = HSS design wall thickness
        B = overall HSS width
        D = round HSS outside diameter
        PHI = 1

       Round HSS   formula (8.2-1).
         Rn = Qf * 5Fy* t^2/(1 + .25N/D)

       Rectangular HSS   formula (8.2-2).
         term1 = Fy * t^2/(1 - t1/B)
         term2 = 2N/B + 4 * SQRT[1 - t1/B]
         Rn = term1 * term2 * Qf

       Load = PHI * Rn

(266) Plate pull-through along bolt line:
    Rl = number of bolts along line of force
    Nl = number of bolt lines parallel to force
    Dh = bolt diameter + .125
    Av = shear area for one bolt line
    Np = number of plates
    Av = 2*(End_dist + (Rl - 1)*S - (Rl - .5)Dh)*t
    (J4-1) shear rupture:  Rn1 = .6 Fu*Av
    (J5-3) shear yield:    Rn2 = .6Av*Fy
  Axial load = MIN[PHI*Rn1, PHI*Rn2]*Nl*Np

    For moment applications --
  Moment = MIN[PHI*Rn1, PHI*Rn2] * Nl * Moment arm

(267) Claw angle net/gross area:
      (W brace, web horiz, web claw L conn)
      Gross width = Leg + Osl - t
      Ag = Gross width * t
      An = (Gross width - Dh) * t
      Rn_g = PHI * Fy * Ag        (J5-1)
      Rn_n = PHI * Fu * An * U    (J5-2)
  Axial load = 4 * MIN[Rn_g, Rn_n]

(268) Claw angle tearout:
      (W brace, web horiz, web claw L conn)
      Ant = (angle_toe_edge - .5Dh) * t
      Agt = angle_toe_edge * t
      Anv = t* S * (Row - 1 ) + Le - (Row - .5) * dh
      Agv = t * S * (Row - 1 ) + Le
      T1 = .6Fy * Agv + Fu * Ant  (J4-3a)
      T2 = .6Fu * Anv + Fy * Agt  (J4-3b)
  Axial load = 4 * PHI * MAX[ T1, T2]

(269) W brace net/gross area:
      (W brace, web horiz, web claw L conn)
      Ag = (D - 2Tf)Tw + 2Bf*Tf + Fillet Area
```

```
(270) V brace, guss shr tab weld to col:
      (guss to col and beam)
      ew = eccentricity of load
      mom = vc * ecc
      f1 = vc / weld_len
      f2 = 0
      f3 = mom * (weld_len / 2) / Ix
      f4 = hc / weld_len
      fr = SQRT[ (f1 + f2)^2 + (f3 + f4)^2 ]
      Fr = PHI * 0.707 * 0.6 * Fexx * Eff_weld
      load = 2 * Fr / fr

(271) H brace guss to L bolt shr
      (guss clip L to bm web conn)
      ex = eccentricity in the x direction
      ey = eccentricity in the y direction
      Rv = Fv * Ab
      num = Row * Column
      dx = Spa * (Row - 1) / 2
      dy = Col_spa * (Column - 1) / 2
      px = P * cos(Phi)
      py = P * sin(Phi)
      mom = abs( px * ey - py * ex )
      fx = px / num + mom * dy / Ip
      fy = py / num + mom * dx / Ip
      fv = SQRT( fx^2 + fy^2 )
      Axial load = maximum P to satisfy fv <= Rv

(272) H brace guss to L bolt brg:
      (guss clip L to bm web conn)
      ex = eccentricity in the x direction
      ey = eccentricity in the y direction
      Rv = PHI * Fp * Db * Tf
      num = Row * Column
      dx = Spa * (Row - 1) / 2
      dy = Col_spa * (Column - 1) / 2
      px = P * cos(Phi)
      py = P * sin(Phi)
      mom = abs( px * ey - py * ex )
      fx = px / num + mom * dy / Ip
      fy = py / num + mom * dx / Ip
      fv = SQRT( fx^2 + fy^2 )
      Axial load = maximum P to satisfy fv <= Rv

(273) H brace guss to L weld:
      (guss clip L to bm web conn)
      length = angle length
      side = 1 for a one side clip
           = 2 for a two sided clip
      k = Leg - Setback
      Weld_len = side * (length + 2 * k)
      Fr = PHI * 0.707 * 0.6 * Fexx * min(Eff_weld, Weld_size)
      Axial load = Fr * Weld_len
```

(274) H brace guss L, bolt brg on bm web:
(guss clip L to bm web conn)
coef = bolt group center of rotation
Rv = bearing tearout strength
side = 1 for a one side clip
    temp = Shear * coef * Row * Rv
side = 2 for a two sided clip
    temp = 2 * Shear * Row * Rv
Axial load = temp / cos(Phi)

(275) H brace guss to L weld:
(guss clip L to bm web conn)
x0 = angle leg + bm tw / 2
y0 = brace wp to centerline of angle
length = angle length
side = 1 for a one side clip
     = 2 for a two sided clip
wx = Leg - Setback
wy = length
x bar = (wx)^2 / Weld_len
y bar = wy / 2
Ix = (wy)^3 / 12 + 2 * wx * (y bar)^2
Iy = (wx)^3/6 + 2*wx*(.5*wx-x bar)^2 + wy*(x bar)^2
Ip = Ix + Iy
m = (1/Weld_len + y0 * wy / (2 * Ip))^2
k = (1/Weld_len + x0 * (wx - x bar)/(2 * Ip))^2
denom = sin(phi)^2 * m + cos(phi)^2 * k
Rv = PHI * 0.707 * 0.6 * Fexx * min(Eff_weld, Weld_size)
Axial load = side * SQRT((Rv)^2 / denom)

(276) H brace guss gross area interactive stress:
(guss clip L to bm web conn)
Refer to misc design note 32;
x0 = angle leg + bm tw / 2
length = angle length
Ag = length * Tf
Sx = Tf * (length)^2 / 6
tau = cos(Phi) / Ag
sigma = sin(Phi) / Ag+ cos(Phi) * x0 / Sx
Axial load = maximum to satisfy the yield criterion

```
(277) H brace guss net area interactive stress:
      (guss clip L to bm web conn)
      x0 = angle leg + bm tw / 2
      length = angle length
      net length = length - (row - 1) * dh
      An = net length * thick
      Fv = PHI * 0.6 * fy
      Ft = PHI * fy
      t1 = cos(Phi) / (An * Fv)
      t2 = sin(Phi) / (An * Ft)
      t3 = cos(Phi) * x0 / (Sx_net * Ft)
      Axial load = shear / ( t1 + t2 + t3 )

(278) H brace guss L to bm bolts, shr/ten:
      (guss clip L to bm web conn)
      num = Row * Column
      b = gage - Tf
      if Column = 2
          fv = P * cos(Phi) / (num * Ab)
          ft = tension per bolt with prying
      if column = 1
          f1 = P * cos(Phi) / Row
          mom = P * cos(Phi) * gage
          f2 = mom * .5 * Spa * (Row - 1)/Ip
          Rv = SQRT( f1 * f1 + f2 * f2 )
          fv = Rv / Ab
          ft = tension per bolt with prying
      Fv = allowable bolt shear
      Ft = allowable bolt tension
      Axial load = max P to satisfy fv <= Fv & ft <= Ft

(279) H brace guss L OSL bending with prying:
      (guss clip L to bm web conn)
      (AISC 'Engr. Journal', Vol. 22, No 2, page 65
       and 9th ed AISC, pg 4-90 )
      Tf = Cn_thick
      p = Cn_depth/Row   d'= Db+1/16 (Db+2 mm)
      Delta = 1-d'/p     M = p*Tf^2*Fy/4.44
      T = (1 + Delta) * M / b'
      Tension = Column * Row * T

(280) Bolted moment connection with HSS column:
      (Flange plate Weld to Column)
      Col_face = Width of HSS column - 2 * Corner Radius
      Con_depth = 3/4 HSS column depth - Corner Radius
      Tf = Thickness of the flange plate
      Eff_weld_plate = PHI * 0.6 * Fy * Tf / (PHI * 0.707 * 0.6 * Fexx * 2)
      Eff_weld_column = PHI * 0.6 * Fy * Tw / (PHI * 0.707 * 0.6 * Fexx)
      Eff_weld = min( Eff_weld_plate, Eff_weld_column)
      Fr = 2 * 0.707 * Eff_weld * 0.3 * Fexx
      Fr_perp = 1.5 * Fr * Col_face
      Fr_para = 2 * Fr * Con_depth
      Moment = (Fr_perp + Fr_para) * (D + Tf)
```

```
(281) Bolted moment connection with HSS column:
      (Flange plate Tension / Compression)
      Pw = Total flange plate Width
      Col_width = width of HSS column
      Tf = Thickness of the flange plate
      Ws = 0.5 * (Pw - Col_width - 0.125)
      Ag = Ws * 2 * Tf
      Moment = Qs * PHI * Fy_plate * Ag * (D + Tf)

(282) Axially load shear tab fillet welds:
       ft1 = applied axial load / ( 2 * conn_depth)
       ft2 = R * ew * .5 * conn_depth / Ix;  twisting
       fv = R / (2 * conn_depth )
       fr = SQRT( fv^2 + (ft1 + ft2)^2 )
       Fr = PHI * Fw * Aw,      Table J2.5
      Load = largest R to satisfy  fr <= Fr

(283) HSS col wall strength:
      (Beam to column welded moment connection)
       HSS Connections Manual, Specification 8.1
      Moment = (D - Tf) * Load result from formula 264

(284) End pl strength, HSS beam with axial load:
      (HSS Manual page 6-40 thru 43)
         p = plate length tributary to each bolt
         a = dim from bolt line to side face of HSS
         b = plate horiz edge dist; t1 = pl thickness
         n = number of bolts
         PHI_rn = design tensile strength of one bolt
         Dh = Bdia + 1/8
         a' = a + .5 * Dh;    b' = b - .5 * Dh + t
         delta = 1 - Dh / p
         rut = Pu / n; external tensile load on one bolt
         K = 4*b'/(PHI * Fy_plate * p);   PHI = .9
         term1 = K * PHI_rn / t1^2  - 1
         term2 = (a + .5 * Dh)/(delta(a + b + Tw ))
         alpha = term1 * term2  >= 0.
         PHI_Rn = t1^2 * ( 1 + delta*alpha)n / K
       Axial load = PHI_Rn

         Bolt tension including prying:
         rut *( 1 + (b'/a')*( alpha / (1 + alpha))
         where  alpha = K * (rut / t1^2 -1)
```

```
(285) End pl weld strength, HSS beam with axial load:
      (HSS Manual page 6-40 thru 43)
         H = HSS section depth
         fh = applied ten / ( 2.* H )
         Fw = .6Fexx
         When tension predominates, dsn shr/dsn ten <= .15:
         Fw = .6Fexx( 1.+.5 SIN^1.5(theta)),  theta = 90 deg
         Fr = PHI * Fw * .707 * weld size
       Load = 2 * H * SQRT( Fr^2 + fh^2 )

(286) End plate bolt shear strength, HSS beam with axial load:
      (HSS Manual page 6-40 thru 43)
       Tb = applied tension per bolt with prying
            from formula 284
         Slip-critical bolts J-8a:
          rstr = 1.13*mu*Tb Ns
       Load = PHI*rstr * number of bolts

         Bearing bolts J3-7:
          ft = Tb / Ab
          fv = max value from interaction eqns, Table J3.5
       Load = fv * Ab * number of bolts

(287) Block shear rupture strength, with applied axial tension:
       AISC specification section J4.3
          t1 = thickness required for axial tension, (J4-3a & -3b)
          t2 = thickness required for shear   (J4-3a & -3b)
          Elliptical interaction.....
          Reqd T = SQRT( t1^2 + t2^2 )
       Load = Applied shear * conn_thick / Reqd T
```

```
(288) Flush and Extended Moment End Plate connection strength:
      Refer to AISC design guide 16, analysis flow chart, for PHI
      factors, definitions and calculation of variables.

         Plate yielding:
               PHIb Mpl = PHIb * Fypl * tp^2 * Y
         Bolt rupture, no prying:
               PHI Mnp = PHI[ 2Pt * SUM(dn)]

         PHI Mnp < PHIb Mpl / 1.11
            PHI Mn = min( PHI Mnp, PHIb Mpl / gamma r )
            Thick PL behavior controlled by bolt rupture

         PHI Mnp >= PHIb Mpl / 1.11
            PHIb Mpl / gamma r < PHI Mq
            PHI Mn = PHI Mq;  Thin PL behavior controlled by bolt rupture

            PHIb Mpl / gamma r > PHI Mq
            PHI Mn = PHIb Mpl / gamma r;  Thin PL behavior controlled by PL yieldin

     Moment = PHI Mn

(289) Shear tee flange bolts, shear/tension interaction:
      rigid support, LRFD 3rd ed page 7-11 and example 10.15
        n = number of bolts, n_prime = num of bolts above the NA
        d_m = moment arm between the resultant ten and comp force
        Pu = connection shear force
        e = eccentricity of Pu from the tee flange
        r_uv = Pu / n
        r_ut = Pu e / 2(n_prime * d_m)
        PHI r_n = PHI Ft * Ab
                  Ft calculated from interaction formulas sec J3
      Load = maximum Pu for PHI r_n = ru_t

(290) Shear tee flange strength, bending with prying:
      rigid support, LRFD 3rd ed page 7-11 and example 10.15
        n = number of bolts, n_prime = num of bolts above the NA
        d_m = moment arm between the resultant ten and comp force
        Pu = connection shear force
        e = eccentricity of Pu from the tee flange
        p = conn length / number of bolt rows
        See 'prying action' page 9-10 for definition  of p,
        b_prime, delta, alpha_prime and t_reqd

        r_ut = Pu e / 2(n_prime * d_m)
        t_req = SQRT(4.44*r_ut*b_prime/(pFy(1 + delta*alpha_prime)))
      Load = Maximum Pu for t_reqd = Tf
```

```
(291) Horizontal clip angle with axial load, web bolt shear:
      eccentricity not considered
        Bearing type bolts:
          Axial Load = PHI * Fn * Ab * Row * Column * Shear
        Slip-critical bolts at factored loads:
          Axial Load = PHI * Rstr * Row * Column * Shear

(292) Horizontal clip angle with axial load, bolt bearing:
      eccentricity not considered
        N_e = number of edge bolts
        Lc = Le - .5Dh
        Rn_e = Rn     (J3-2a) or (J3-2c)

        N_i = number of interior bolts
        Lc = Spacing - Dh
        Rn_i = Rn     (J3-2a) or (J3-2c)

      Load = PHI(Rn_e * N_e + Rn_i * N_i) * Shear

(293) Col. web local bending stress (axially loaded hoiz. clip L):
      (Tw, Depth & K_dist are column dimensions)
        c = Distance between the top & bot bolts
        l = c + 12 * Tw;  effective web length
        For a 1 inch wide strip of web, with fixed ends:
          Sx = SQR[Tw] / 6
          L = Depth - 2*K_dist,        length of 1 inch beam strip
          d = spacing * ( row - 1 ),   length of distributed load
          a = .5 * ( L - d )
          P = 12 * PHI * Fy * Sx * L / ( L^2 + 2aL - 2a^2 )
        Axial load = P * l

(294) Horizontal beam web to clip L weld stress:
        side = 2 for a two sided clip
        Min_web = .707 * PHI * Fw * Weld_size * side / Fv_web
        Web_factor = MIN[ Tw / Min_web, 1 ]
        Fr = PHI * .707 * Fw * Weld_size
        L = conn_depth + 2 * ( conn_width - conn_stbk )
        Axial Load = Fr * L * side * Web_factor

(295) Horizontal clip L to supporting web weld stress:
      (if 2-sided, only the bottom angle carries load)
        Min_web = .707 * PHI * Fw * Weld_size / Fv_web
        Web_factor = MIN[ Tw / Min_web, 1 ]
        Fr = PHI * .707 * Fw * Weld_size
        L = conn_depth + 2 * osl
        Axial Load = Fr * L * Web_factor
```

```
(296) Interactive web stress, horizontal clip L with axial load:
        See design note #22.
        Tension load ---
          Welded angle:
            Coped top AND bottom -
              A1 = tw * conn_depth
              A2 = 0
            Coped top OR bottom -
              A1 = tw * conn_depth
              A2 = tw * ( conn_width - setback )
            Uncoped -
              A1 = tw * conn_depth
              A2 = 2 * tw * ( conn_width - setback )
            Rv = 0.54 * Fy * ( A1 + A2 )
            Rt1 = 0.54 * Fy * A2 + 0.75 * Fu * A1 (shear yield, tension fracture
            Rt2 = 0.45 * Fu * A2 + 0.9 * Fy * A1 (shear fracture, tension yield
            Rt = MAX[ Rt1, Rt2 ]
          Bolted angle:
            Coped top AND bottom -
              A1 = tw * conn_depth
              A2 = 0
              A3 = tw * ( row - 1 )( spacing - Dh )
              A4 = 0
            Coped top OR bottom -
              A1 = tw * conn_depth
              A2 = tw * ( conn_width - setback )
              A3 = tw * ( row - 1 )( spacing - Dh )
              A4 = tw * [( Lh - 0.5 * Dh ) + ( column - 1 )( col_spa - Dh )]
            Uncoped -
              A1 = tw * conn_depth
              A2 = 2 * tw * ( conn_width - setback )
              A3 = tw * ( row - 1 )( spacing - Dh )
              A4 = 2 * tw * [( Lh - 0.5 * Dh ) + ( column - 1 )( col_spa - Dh )]
            Rv = 0.54 * Fy * ( A1 + A2 )
            Rt = 0.75 * Fu * A3 + 0.45 * Fu * A4
          t_check = Rv * SQRT[ 1 - ( Ten load / Rt )^2 ]
        Compression load ----
          Coped top AND bottom -
            A1 = tw * conn_depth
            A2 = 0
          Coped top OR bottom -
            A1 = tw * conn_depth
            A2 = tw * ( conn_depth - setback )
          Uncoped -
            A1 = tw * conn_depth
            A2 = 2 * tw * ( conn_depth - setback )
          Rv = 0.45 * Fy * ( A1 + A2 )
          Welded angle:
            A3 = tw * conn_depth
          Bolted angle:
            A3 = tw * ( row - 1 ) * spacing
          Rc = 0.9 * Fy * A3
          c_check = Rv * SQRT[ 1 - ( Comp load / Rc )^2 ]
       Load = MIN[ c_check, t_check ]
```

LRFD CONNECTION DESIGN NOTES

(1)Design calculation information/warning messages:
Certain conditions will cause informational messages to be displayed on the design calculation printout sheet. These conditions have not caused a connection to fail but nevertheless may have an impact on the design or function of the connection.

These messages are intended to call attention to local connection conditons that may need to be evaluated by the engineer who is in the best position to determine how the structure and connections are intended to function and evaluate the effects of the connection design on the structural system.

(2)Connection elements or geometry not meeting the applicable AISC design criteria, require engineering evaluation as to their suitability for the intended application.

(3)Design calculation printout sheet:
Only the conditions listed under the 'LIMIT STATE' heading will cause a connection to fail. Other allowable loads may be shown to provide additional information for the designer.

(4)Bolts:
Strengths of connections using bearing type bolts are designed for comparison with factored loads in accordance with AISC specification section J3.6

Strengths of connections using slip-critical type bolts are designed for comparison with factored loads in accordance with AISC specification sections A-J3.8 and A-J3.9.

(5)Connection ductility checks:
Double clip angle, shear end plate, shear tee
Reference: LRFD Manual, Vol II pg 9-170

```
Conn. bolted to the supporting member ---
    Min A325 bolt diameter to preclude bolt fracture.
    db_min = .163 * Tf * SQRT[ (Fy/b) * K ]
          <= .69 * SQRT[ ts ]
    b = dim from hole gage line to k dist
Tee/Clip L  Welded to the supporting member ---
    Min E70 weld size.
    w_min = .0158 * Fy * SQR[Tf]/b * K
          <= 75 * ts
    b = dim. from  k1 dist to toe of flg

L = connection depth
K = SQR[b] / SQR[L] + 2
ts = tee stem thicknes
Tf = tee flg or angle OSL thicknes
```

```
 (6)Moment connection to W column flange:
         Design strength P1, P2, P3 from calculations 83, or 113.
         Design strength P4 from calculation 25.
         P1 = Col web yielding  at bm ten or comp flange, (K1-2)
         P2 = Col compression web buckling, (K1-8)
         P3 = Col local tension flange flexure, (K1-1)
         P4 = Col web crippling, (K1-4, -5a, -5b)
         PHI*Rn = MIN[ P1, P2, P3, P4 ]

         When PHI*Rn > concentrated flange force,
         stiffeners are required.
         Stiffener design:
            Ast = (Concentrated flg force-Pn)/Fy_stiff
            Fy_stiff = 36
            Fy_col <= 36; Stiffener b/t <= 95/SQRT[Fy_stiff]
            Fy_col >  36; Stiffener b/t <= 65/SQRT[Fy_stiff]

(7)Prying action for tension type connections:
          (Volume II, page 11-6 thru -11)
           a = Edge distance of fitting <= 1.25*b
             (For heavy clip angles with 2 cols of bolts, a is
              1.5 inches from the inside bolt col.
              in formulas 27, 40, and 43 )
           b = Bolt C/L to face of fitting or web
           T = Axial tension force/Row/Column, (applied tension/bolt)
           B = Allow. tension per bolt, PHI*Rn kips
           a'= a+.5*Db,             b'= b-.5*Db
           d'= Db+1/16  (Db+2 mm),  rho = b'/a'
           p = Cn_depth/Row <= conn gage,        Delta = 1-d'/p
           tc = SQRT[8*B*b'/(p*Fy)]
           alpha = (1/delta)*[(T/B)/(t/tc)^2-1];  0 <= alpha <=1.
           alpha' = [1/(delta*(1+rho)]*[(tc/t)^2-1]

          Design:
             Q = B*delta*alpha*rho*(t/tc)^2
             treqd = SQRT[(8*T*b')/(p*Fy*(1+delta*alpha'))]

          Analysis (page 11-11):
             1. alpha' > 1.,       T_allow = B*(t/tc)^2)*(1+delta)
             2. 0 <= alpha' <= 1., T_allow = B*(t/tc)^2*(1+delta*alpha')
             3. alpha' < 0,        T_allow = B

             1 is plate failure mode, 2 is bolt failure mode
               with prying, 3 is bolt failure mode without prying
             treqd = SQRT[8*T*b'/(p*Fy)], thick. to eliminate prying force.
```

(8)Definition of a Rigid support:

Beam stiffness = Ix / Span Length
HSS/Pipe col stiffness = I / 120; average 10-0 story ht
The distance between beam end points must be less than
or equal to 2.5 * Tw, the supporting member web, or 2 in. max.

1.Beam to a girder web with a beam framing opposite,
  and beam opposite stiffness >= beam stiffness.
2.Beam to W col flange, with Col Ix >= 2 * Beam Ix.
3.Beam to W col flange, with tw_col >= tw_beam and
  and beam opposite stiffness >= beam stiffness.
4.Beam to W col web, with Col Iy >= 2 * Beam Ix
  and col T dist / Tw <= 24
5.Beam to W col web, with a beam framing opposite
  and beam opposite stiffness >= beam stiffness.
6.Beam to a HSS or Pipe column; column not a slender wall
  section & col stiffness >= beam stiffness.

These definitions are not part of the AISC procedure which
states: 'The rigidity of the support is left up to
engineering judgement' - Vol II page 3-103.

(9)Single-plate shear connection:

```
 See (25) for extended shear tab design procedures.
One bolt column, 2 to 12 bolt rows
Standard round or short slotted holes
3 in. vertical bolt spacing
(SDS/2 accepts 2 3/4 to 3 1/4 inclusive)
1 1/2 in. vertical edge distance, Lv
1 1/2 in. minimum horiz edge dist, Lh
PL thickness <= .5 * Db + 1/16 in.
             >= L*SQRT(Fy)/(234K)
             K = PL buckling coefficient
             >= 1/4 inch
Fy = 36 ksi for PL.    All grades of beam steel
Nominal weld metal tensile strength, Fexx = 70 ksi
Composite or non-composite beams, all loadings
Valid dihedral angle: 20 to 90 degrees.
A325 or A490 bolts only.
Perp. fillet welds: weld size  >= 1.38 * t * Fy / (.95 * Fexx)
                               >= .75t, for A36 pl material
Skew fillet welds:  eff throat >= eff throat for perp fillets.
CJP weld:           eff throat  = t
(For these conditions, the weld is not the critical
connection element since the plate yields before
the weld.)
For A > 3.5 in. the net plate section bending
stress is checked.
For an axial load, the net & gross  pl area interactive
stress is checked, using eb or ew:
   fv/Fv + fb/Fb <= 1
Stabilizer bars:  b/t <= 95/SQRT[Fy] and l/r <= 120

For a connection with more than one bolt column, dim. 'a' is the
distance from the weld line to the CG of the bolt group.
This is not a part of the AISC design procedure and will
occur only on a 'user' connection.

For a supporting HSS column wall or beam web, the maximum
plate thickness to prevent punching shear failure is:
1.2 * PHI_v * Fu_HSS * tw_HSS / (PHI_t * Fy_pl)
HSS Manual, spec equation (9.3-3)
The supporting HSS column wall or W web, strength
requires evaluation for an axially loaded shear tab. The
design of this connection for axial load is not a part
of the AISC design procedure.
A 'thin-walled' HSS col. has b/t > 238/SQRT[Fy]
where b = section depth - 3*Tw, AISC B5.1.
```

(10) Bolt and weld design eccentricities:
(Volume II Appendix C, pages C-1 & C-2; LRFD 3rd ed, pg 10-112
Flexible support & std holes: eb = MAX[| Row - 1 - a |, a]
Rigid support & std holes: eb = |Row -1 - a |
Flexible support & short slots: eb = MAX[| 2 * Row / 3 - a |, a]
Rigid support & short slots: eb = | 2 * Row / 3 - a |
(Rigid supports when a > 3.5; eb will be the
the max of eb and a, and ew = eb)
ew = eb + a; (a = dim. from weld line to bolt group)

(11) Extended end plate moment connection:
Web weld is sized to develop the the beam web bending stress.
Eff. web weld length = dist from inner row of tension bolts +
2 * Bdia to the inside face of the compression flg. or beam
C/L to inside face of compression flg, whichever is smaller.

Un-stiffened four-tension bolt type:
1. A325 or A490 bolts, 3/4 to 1 1/2 dia.
2. Interactive stress checked for tension bolts in
in bearing type connections.
3. Horizontal gage <= beam tension flg width.

Stiffened eight-tension bolt type:
1. End plate material must be A36, Fy = 36 ksi.
2. Beam section must be hot-rolled and included in the
AISC Allowable stress Design Selection Table.
3. Pf must not exceed 2 1/2 inches.
Vertical spacing of tension bolts >= 3 * Db,
except 3 in. may be used for Db < 1 in.
4. Horizontal gage 5 1/2 to 7 1/2 inches.
5. Triangular stiff. thickness >= Tw
6. A325 bolts only, 3/4 to 1 1/2 diameter.
7. Interactive stress checked for tension bolts in
bearing type connections.

When this connection frames to a W column web, the supporting
web strength needs to be evaluated.

(12) Shear connection design notes:
(Clip angle, Bent plate, Single-plate, Thru-plate, Tee,
Web plate, and End plate)
1. The connection depth will be >= half the supported
beam 'T' distance.
2. Valid dihedral angle for bent plates:
Two side conn or plate on acute angle side
only, 56 to 90 degrees.
Plate on obtuse angle side only 18 to 90 degrees.
3. Valid dihedral angle for shear tabs:
20 to 90 degrees.
4. The weld for a shear tee is designed to develop Ro.

5. Shear tee design, AISC Vol II page 3-102:
   One bolt column, 2 to 7 bolt rows
   Std holes or short slots only
   Valid for A325 & A490 bolts only; 3 in. spacing.
   (SDS/2 accepts 2 3/4 to 3 1/4 inclusive)
   Field bolts: Flexible support: eb = La
   Rigid support: eb = 0
   Shop bolts: Flexible support eb = 0
   Rigid support eb = la_x
   Shop weld:
   Flexible support: ew = 0, Rigid support: ew = La
   Bf/2Tf >= 6.5, Db/Tw >= 2.0
   (Tw/Db) / (Tf/Tw) approx. = .25
   Weld metal nominal tensile str. >= 60 ksi

6. Connections designed for axial load:
   a. Non-moment end pl.
   b. Extended end pl moment conn.
   c. Two side clip angle -- except angles welded
   to the supporting member.
   d. Single-plate shear connection
   'Full depth' connections are designed.
   e. bolted moment beam splice.
   Since Rbs is checked using only the shear load the
   combined beam web stress is limited by calculation 169.
   Local supporting web stress calculated by formulas 174, 175
   & 176 will not fail a connection, but the limiting axial
   load is shown on the design calculation print-out.

7. One-side (eccentric) clip angle:
   Design is valid for Gages limited to the values
   recommended in AISC 'Vol II Connections'
   Figure 3-7; 2 1/4 to 3 in. (57 - 76 mm).
   Designs for connections using larger gages require
   engineering evaluation.
   Design is valid only for standard holes in the leg of a
   clip angle bolted to a supporting member.

8. Web dblr. dsgn. valid only for Clip Ls or Bt pls
   on bms with perp end cuts and no axial load.

9. Thru plate, pipe or tube column:
   (Not an AISC procedure)
   Maximum of 3 bolt columns.
   Bolt eccentricity = La.
   Seven or fewer rows, weld eccentricity = MAX[La, row]
   More than 7 rows, weld eccentricity = La
   (For connections with La <= 3, one bolt column,
   7 or fewer bolts, and tp <= .5 * Db + 1/16, .
   the maximum weld size = .75 * tp)

```
(13)Vertical brace gusset connection interface forces:
           AISC Uniform Force Method.
           (Applies to formulas 147 thru 157)
           eb = dim from brace WP to beam flange
           ec = dim from brace WP to face of column
           theta = angle between brace and column
           alpha_ = horiz dim from face of clips to ctr of guss conn
           beta_ = vert dim from beam flg to ctr of gusset conn
       Gusset edge at beam (welded) is the more rigid connection:
           alpha = eb * TAN(theta) - ec + beta * TAN(theta)
           beta = dist from bm flange to ctr of clip conn
       Both gusset edge connections (clip angles) are equally stiff:
           alpha = {K'*TAN(theta)+K*SQR[alpha_/beta_]}/D
           beta = (K' - K * TAN(theta))/D

         K' = alpha_*(TAN(theta) + alpha_/beta_)
         D = SQR[TAN(theta)] + SQR[alpha_/beta_]
         K = eb * TAN(theta) - ec
         Pu = axial brace force
            (Hu = horiz component, Vu = vert. component)
         r = SQRT[(alpha + ec)^2 + (beta + eb)^2]
         Hub = alpha * Pu / r,     Vub = eb * Pu / r
         Mub = Vub * |alpha - alpha_|
             = 0 when |alpha - alpha_| <= .5
         Huc = euc * Pu / r,        Vuc = beta * Pu / r
         Muc = Huc * |beta - beta_ |
         The uniform force method requires that Vub and Huc
         be included in the beam-column connection design.

         AISC special cases:
           Special case 1
             Connection to a col flange:
               The couples Mub = Hub*eb and Muc = Vuc*ec/2  are considered
               to have been included in the column and beam design.
               See Vol II pages 7-154 and 7-155 for discussion.
             Connection to a col web:
               The couple Mub = Hub*(.5*Db) is considered to
               to have been included in the beam design.
               See Vol II page 7-143 for discussion.

           Special case 2, minimizing shear in the beam-to-col conn.:
               Vub is reduced by Delta_Vub
               Guss to col interface:  Vuc + Delta_Vub
               Mub = Delta_Vub * alpha
               See LRFD Vol II page 11-22 for discussion.

(14)Bent plate connections:
         Bending is checked for both legs of one-side connections.
         Design is valid only for standard holes in the leg to
         the suppporting member of a one-side conn.

(15)Double L and C brace stitch plates:
         Stitch plate spacing is calculated using the overall length of
         the brace, in accordance with AISC sections D2 & E4.
```

```
(16)Notes for 'heavy' clip angles, two bolt columns in each leg:
          The shear strength is reduced only for the inside bolt columns
          nearest the beam web. Only these bolts are assumed effective
          in resisting the beam's axial tension force.
          Eccentricity is considered in the design of web bolt brg.
          and shear stress, using the elastic method of analysis.

(17)Seated connections:
             Beam to seat connection made using A325 or A490 bolts.
             For stiffened seats, the stiffener is finished to
             bear on the seat plate or angle leg.
             Weld return length = .2 * stiffener depth for all
             plate and tee seats.
             Stiffened PL & Tee seat design procedure is valid for:
        Seats on W col webs --
          1. 'Standard' column sections
          2. Seat pls not welded to col flanges
             (Welding seat pls to the col flanges may  induce
             moments into the column cross section.)
          3. Beam to seat connection  bolts located <=.
             the maximum of .5*seat width or 2 5/8
             inches from the face of the web.

(18)Definition of a standard W column section:
              1. T / tw <= 36.1
              2. Nominal depth <= 14
              3. d * tw^3 / ( bf * tf^3 ) <= .362
              4. Wt per ft <= 730

(19)Welding:
          Reference code: AWS D1.1-96 Structural Welding Code

          Maximum  effective fillet weld sizes:
            Fv = PHI * .6 * Fexx
            t = appropriate supported or supporting member thickness
            Vw = coefficient tabulated below
            Eff_weld = PHI * .6 * Fy * t / (.707 * Fv * Vw)

            Two side clip L, Bent pl, or End shear pl to a
            supported beam web, Vw = 2
            One side clip L or Bent pl to supported beam web, Vw = 1
            Seats on opposite sides of a W col web,Vw = 2
            Seats on a W col flg, or web without a member opposite
            or Seats to a tube column, Vw = 1
            Brace gusset pl to beam or col, Vw = 2

          Effective throat for skew fillet welds:
             theta1 = Acute dihedral angle
             theta2 = 180 - Theta1
             Weld1 = weld size on acute angle side
             Weld2 = weld size on obtuse angle side
             T1 = .5 * Weld1 / SIN(.5 * theta1) - Z loss reduction
             Z loss = 1/4, Theta1 < 45;  1/8, 45 < Theta1 < 60 degrees
             Alpha = Theta2 - 90,    theta = 90 - theta2 / 2
             K = t * TAN(Alpha) * SIN(Theta)
             T2 = .5 * Weld2 / SIN(.5 * thetai2) - K
             Eff_throat = T1 + T2
             (90 deg dihedral angle: Eff_throat = SIN(45) * Weld1 * 2)
```

Fillet weld strength per inch: 1.392 * D (table J2.5)
D = # of sixteenths in weld size.
Fexx = 70, PHI = .75, Rn = .6*Fexx*Aw

Complete joint penetration welds:
CJP welds may be designed for a beam flange at an extended moment end pl, flange plate moment connection to a W column, column flange stiffener, shear tab or a skew end plate. Matching weld metal must be used for these welds. See AISC Table J2.5 & AWS D1.1, Table 3.1

Single-plate shear connection weld types:
Dihedral angle = 90 degrees: perpendicular fillets
90 > Dihedral angle >= 60 degrees:
Skew fillets with a Rn1 <= 3/16 in.
TCU-4b with a Rn1 exceeding 3/16 in.
(AWS D1.1, figure 3.11)
60 > Dihedral angle >= 45 degrees: TCU-4b
Prequalified
45 > Dihedral angle >= 20 degrees: TCU-4a
Not prequalified
20 degrees > Dihedral angle: not allowed.

(20) Transverse beam stiffeners:
Stiffeners are designed in accordance with AISC K1.9. Stiffener welds are the AISC minimum size fillets and the stiffener ends must be finished to bear on the loaded flange.

(21) Notes on formulas 126, 169 and 203:
These equations check the combined stress on an element where the maximum normal and shear stress occur at the same time; a situation not covered by the AISC specification. SDS/2 uses a linear interaction equation:
fv/Fv + fb/Fb <= 1, or fv/Fv + ft/Ft <= 1.
(There is a brief discussion of this in 'Engineering Journal' third quarter, 1986)

(22) Extended clip angle connection:
An extended clip angle may be used on a beam without an axial load and with a square end cut. The yield strength of the web pl extension must match the yield strength of the beam, and a CJP weld must be used at the junction of the pl and beam.

(23) Clip angles shop welded to a supporting HSS col:
When the toe of the leg attached to the column extends past the column the flare-bevel-groove shop weld at the tube radius corner must provide an effective throat dimension equal to the fillet weld size shown on the detail and design calculation.

(24)HSS Design wall thickness:
The design wall thickness is taken as .93 times the nominal wall thickness for HSS sections not manufactured by the submerged arc welding process: see the Specification for the Design of Steel Hollow Structural Sections, section 1.2.1 and commentary.

(25)Extended shear tab design:
Design of Extended Shear Tabs, AISC final report, Oct 2002
Two to 10 bolt rows, one bolt column
Bolt line >= 2.5, <= 3.5 in. from the toe of the supporting mbr flange
A325 or A490 bolts, STD or SSL holes.
Stability PLs at top and bottom of a tab to a col web
Shear tab extended & welded to the top flg of a supporting bm.
Fillet weld size >= .75 * plate thickness

Dim a = distance from bolt line to centroid of weld group.
Eccentricity:
For n <= 6; eb = n <= Dim a
For n > 6; eb = 3 + n/2 <= Dim a
ew = (Dim a - eb - x_bar_weld

(26)Column web panel-zone shear:
Total shear, Sigma Fu <= PHI Rv, (C-K1-3)
dc = column depth

Min Tw = Sigma Fu / (PHI * .6Fy * dc)

(27)Concentrated forces on HSS
Formulas 264 and 265 are from the Specification for the Design of Steel Hollow Structural Sections, Sections 8.1 and 8.2. The resistance is based on the connection element being centered on the supporting member and Qf is taken equal to 1.0 for all cases; which may be unconservative when there is compression stress in the supporting member.

(28)W Beam or Column web stress under a K brace gusset PL:

Web stress is calculated using the input brace forces.
Theta = the angle between the brace and the member
Ecc = the eccentricity of the brace normal force
measured from the gusset C/L
Normal forces: Tn1, Tn2, Cn1, Cn2 = brace force * Sin(Theta)
Tn1, Cn1 -- ten and comp on the first brace
Tn2, Cn2 -- ten and comp on the second brace
Eff web length = gusset length + 5 * k_dist_bm
A = tw * eff web length, S = tw * (eff web len)^2 / 6
Fp = P/A +- M/S
Fp is the max resulting from combinations:
Tn1 & Tn2, Cn1 & Cn2, Tn1 & Cn2, and Cn1 & Tn2.
M = appropriate force, Tn or Cn * Ecc.
The results shown are the max Fp, tension and comp. stress.

(29)Extended shear end plate for axial loads:

For a tension loaded end plate connection to a column:
The flange-to-plate weld is designed for the flange's portion of the axial load. The web-to-plate weld is designed for the web's portion of the axial load plus the beam end reaction.
Prying action is based on the larger of: the distance from the face of the web to the bolt, and the inside of the flange to the bolt.
The vertical eccentricity from the c.g. of the bolt group to the beam's longitudinal axis is considered for the limit states applying to the bolts, and plate: formulas 27, 60 & 61.
The user must verify that adequate connection rotation capacity is provided for gravity loads.

(30)Safety seat angle:

The angle seat design is based on a load of one-half the beam section weight, with a minimum of .7 kip.
The design limit states are: bolt shear, bolt bearing on the angle, and OSL bending stress;
formula (1) for bolt shear and (28) for OSL bending.

(31)Whitmore effective width section:

The theoretical Whitmore section, at the end of a connection, is determined by spreading the force out at an angle of 30 degrees along both sides of the connection length. The Whitmore section is limited by the edges of the element and may not spread out past these edges. See Fig 9-1 in the third Edition LRFD manual.

(32)Combined stress, Huber-von Mises yield criterion for plane stress:

```
tau = applied shear stress
sigma = applied tension stress
•sigma1 = .5 * sigma + sqrt( .25 * sqr(sigma) + sqr(tau) )
•sigma2 = .5 * sigma - sqrt( .25 * sqr(sigma) + sqr(tau) )
Salmon/Johnson, 'Steel Structures' 3rd edition, formula 2.6.2:
•sqr(Y) = sqr(sigma1) - sigma1 * sigma2 + sqr(sigma2)
substituting:
sqr(Y) = sqr(sigma) + 3 * sqr(tau)
Y = .6Fy for ASD;     .9Fy for LRFD
```

*** END OF COVER SHEET ***