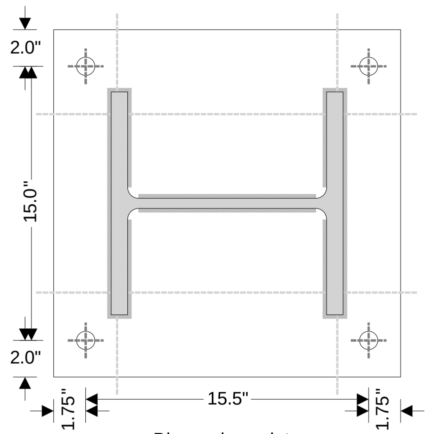
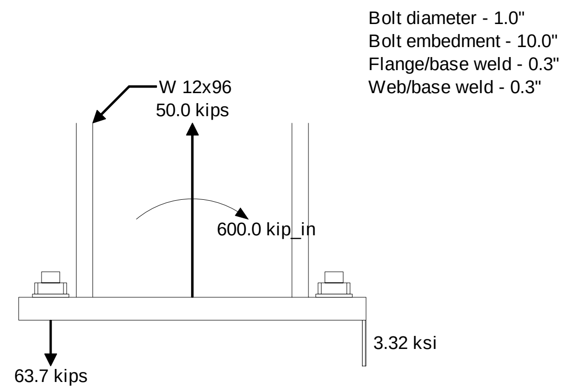
COLUMN BASE PLATE DESIGN (AISC360-05)

AISC 360-10

TEDDS calculation version 2.0.07



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **-50.0** kips (Tension)
Bending moment	M_u = **600.0** kip_in
Shear force	F_v = **0.0** kips
Eccentricity	e = ABS(M_u / P_u) = **12.000** in
Anchor bolt to center of plate	f = N/2 - e_1 = **7.750** in

Column details

Column section	W 12x96
Depth	d = **12.700** in
Breadth	b_f = **12.200** in
Flange thickness	t_f = **0.900** in
Web thickness	t_w = **0.550** in

Baseplate details

Depth	N = **19.000** in
Breadth	B = **19.000** in
Thickness	t_p = **1.250** in
Design strength	F_y = **36.0** ksi

Foundation geometry

Member thickness	h_a = **20.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **30.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **30.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **30.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **30.000** in

Holding down bolt and anchor plate details

Total number of bolts	N_{bolt} = **4**
Bolt diameter	d_o = **1.000** in
Bolt spacing	s_{bolt} = **15.000** in

Edge distance	e_1 = **1.750** in
Minimum tensile strength of steel	F_y = **36** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.65**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N$ = **361.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min})$ = **3600.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **1841.1** kips
Plate bending coefficient X	$X = (4 \times d \times b_f) / (d + b_f)^2 \times \min(1.0, P_u / (P_p \times \phi_c))$ = **-0.04**
Plate bending coefficient λ	$\lambda_l = 1$ = **1.00**
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **3.467** in
Bending line cantilever distance n	$n = (B - 0.8 \times b_f) / 2$ = **4.620** in
Yield line theory cantilever distance n’	$n' = \sqrt{(d \times b_f)} / 4$ = **3.112** in
Maximum bending line cantilever	$l = \max(m, n, \lambda_l \times n')$ = **4.620** in
Maximum bearing stress	$f_{p,max} = 0.85 \times f'_c \times \phi_c \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **3.31** ksi
Maximum bearing pressure	$q_{max} = f_{p,max} \times B$ = **62.98** kips/in
Bearing length - quadratic solution 1	$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 \times P_u \times (f - e))/q_{max})}$ = **34.303** in
Bearing length - quadratic solution 2	$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 \times P_u \times (f - e))/q_{max})}$ = **0.197** in
Bearing length	$Y = \min(Y_1, Y_2)$ = **0.197** in
Tension force in bolts	$T_u = M_u / (N - 2 \times e_1) - P_u / 2$ = **63.7** kips
Max tensile force in single bolt	$T_{rod} = T_u / N_{bolty}$ = **31.9** kips

Base plate yielding limit at bearing interface

Required plate thickness	$t_{p,req} = \sqrt{((4 \times f_{p,max} \times Y \times (l - Y/2))/(\phi_b \times F_y))}$ = **0.603** in

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	$x = abs(l - e_1)$ = **2.870** in
Plate thickness required	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))}$ = **1.080** in

PASS - Thickness of plate exceeds required thickness

Flange weld

Flange weld leg length	t_{fw} = **0.3125** in
Tension capacity of flange	$P_{tf} = b_f \times t_f \times F_y$ = **395.3** kips
Force in tension flange	$F_{tf} = M_u / (d - t_f) - P_u \times (b_f \times t_f) / A_{col}$ = **70.3** kips
Critical force in flange	$F_f = \min(P_{tf}, \max(F_{tf}, 0kips))$ = **70.3** kips
Flange weld force per in	$R_{wf} = F_f / (2 \times b_f - t_w)$ = **2.9** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Nominal weld stress	$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$R_{nf} = F_w \times t_{wf} / \sqrt{(2)}$ = **10.4** kips/in

PASS - Available strength of flange weld exceeds force in flange weld

Transverse web weld

Web weld leg length	t_{ww} = **0.3125** in

Effective width for bending with 45deg distribution $b_{eff} = 2 \times ((N - d)/2 - e_1) =$ **2.800** in

Web weld force $F_{tw} = abs(P_u) \times (A_{col} - 2 \times b_f \times t_f) / A_{col} =$ **11.064** kips

Web weld force per in $R_{wt} = F_{tw} / (2 \times (d - 2 \times t_f)) =$ **0.508** kips/in

Electrode classification number $F_{EXX} =$ **70.0** ksi

Nominal weld stress $\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) =$ **47.250**ksi

Design strength of weld per in $R_{nt} = F_w \times t_{ww} / \sqrt{(2)} =$ **10.4** kips/in

PASS - Available strength of transverse web weld exceeds force in transverse web weld

ANCHOR BOLT DESIGN (ACI318-08)

TEDDS calculation version 2.0.07

Anchor bolt geometry

Type of anchor bolt Cast-in headed end bolt anchor

Diameter of anchor bolt $d_a =$ **1** in

Number of bolts in x direction $N_{boltx} =$ **2**

Number of bolts in y direction $N_{bolty} =$ **2**

Total number of bolts $n_{total} = (N_{boltx} \times 2) + (N_{bolty} - 2) \times 2 =$ **4**

Total number of bolts in tension $n_{tens} = (N_{boltN} \times 2) + (N_{bolty} - 2) =$ **2**

Spacing of bolts in x direction $s_{boltx} =$ **15.5** in

Spacing of bolts in y direction $s_{bolty} =$ **15** in

Number of threads per inch $n_t =$ **8**

Effective cross-sectional area of anchor $A_{se} = \pi / 4 \times (d_a - 0.9743 \text{ in} / n_t)^2 =$ **0.606** in^2

Embedded depth of each anchor bolt $h_{ef} =$ **10** in

Material details

Minimum yield strength of steel $f_{ya} =$ **36** ksi

Nominal tensile strength of steel $f_{uta} =$ **68.4** ksi

Compressive strength of concrete $f'_c =$ **3** ksi

Concrete modification factor $\lambda =$ **1.00**

Strength reduction factors

Tension of steel element $\phi_{t,s} =$ **0.75**

Shear of steel element $\phi_{v,s} =$ **0.70**

Concrete tension $\phi_{t,c} =$ **0.65**

Concrete shear $\phi_{v,c} =$ **0.70**

Concrete tension for pullout $\phi_{t,cB} =$ **0.70**

Concrete shear for pryout $\phi_{v,cB} =$ **0.70**

Steel strength of anchor in tension (D.5.1)

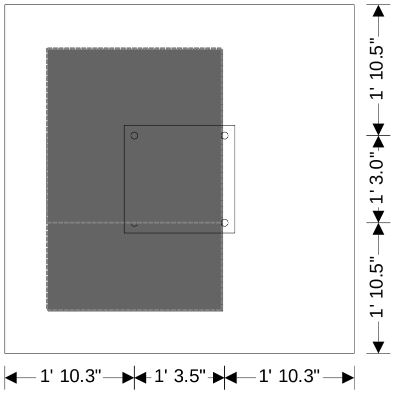
Nominal strength of anchor in tension $N_{sa} = A_{se} \times f_{uta} =$ **41.43** kips

Steel strength of anchor in tension $\phi N_{sa} = \phi_{t,s} \times N_{sa} =$ **31.07** kips

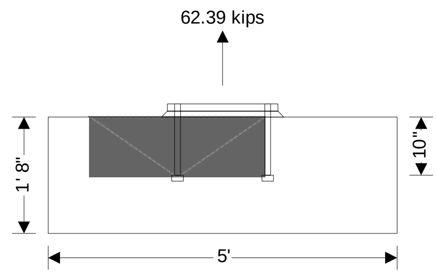
FAIL - Max tension in single bolt exceeds steel strength of anchor

Check concrete breakout strength of anchor bolt in tension (D.5.2)

Tedds Calc	Project				Job Ref.	
	Section				Sheet no./rev. 4	
	Calc. by R	Date 1/6/2012	Chk'd by	Date	App'd by	Date



Plan on foundation



Section A-A

Concrete breakout - tension

Stength reduction factor ϕ_t = **0.65**

Coeff for basic breakout strength in tension k_c = **24**

Breakout strength for single anchor in tension $N_b = k_c \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times h_{ef}^{1.5} \times 1\text{ in}^{0.5}$ = **41.57** kips

Projected area for groups of anchors A_{Nc} = **1350** in^2

Projected area of a single anchor $A_{Nco} = 9 \times h_{ef}^2$ = **900** in^2

Min dist center of anchor to edge of concrete $c_{a,min}$ = **22.25** in

Mod factor for groups loaded eccentrically $\psi_{ec,N} = \min(1 / (1 + ((2 \times e'_N) / (3 \times h_{ef}))), 1)$ = **1.000**

Modification factor for edge effects $\psi_{ed,N} = 1.0$ = **1.000**

Modification factor for no cracking at service loads $\psi_{c,N}$ = **1.250**

Modification factor for uncracked concrete $\psi_{cp,N}$ = **1.000**

Nominal concrete breakout strength $N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **77.94** kips

Concrete breakout strength $\phi N_{cbg} = \phi_{t,c} \times N_{cbg}$ = **50.66** kips

FAIL - Tension in bolts exceed breakout strength

Pullout strength (D.5.3)

Net bearing area of the head of anchor A_{brg} = **1** in^2

Mod factor for no cracking at service loads $\psi_{c,P}$ = **1.400**

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **24.00** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **33.60** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn}$ = **23.52** kips

FAIL - Maximum axial force in a single bolt exceeds pullout strength of single anchor

Side face blowout strength (D.5.4)

As $h_{ef} <= 2.5 \times \min(c_{a1}, c_{a2})$ the edge distance is considered to be far from an edge and blowout strength need not be considered