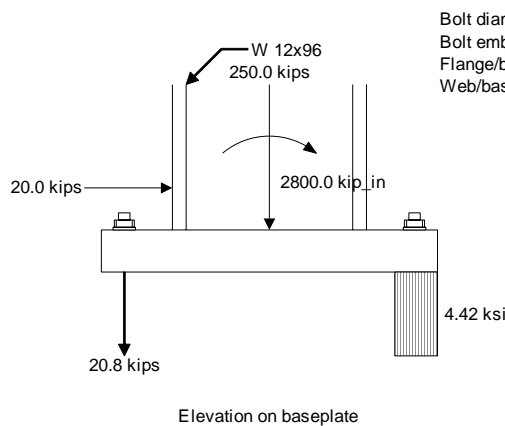
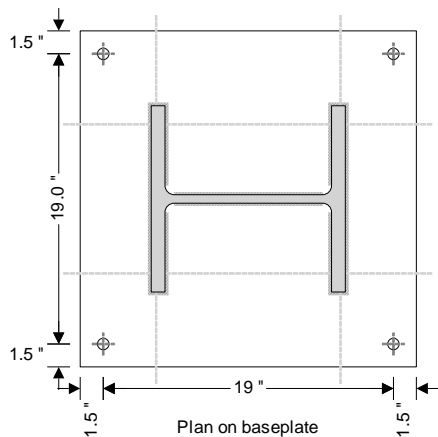


COLUMN BASE PLATE DESIGN (AISC360-05)

In accordance with the LRFD method

TEDDS calculation version 2.0.00


 Bolt diameter - 0.8 "
Bolt embedment - 18.0 "
Flange/base weld - 0.3 "
Web/base weld - 0.2 "

Design forces and moments

Axial force

 $P_u = 250.0$ kips (Compression)

Bending moment

 $M_u = 2800.0$ kip_in

Shear force

 $F_v = 20.0$ kips

Eccentricity

 $e = \text{ABS}(M_u / P_u) = 11.200$ in

Anchor bolt to center of plate

 $f = N/2 - e_1 = 9.500$ 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

 $N = 22.000$ in

Breadth

 $B = 22.000$ in

Thickness

 $t_p = 2.750$ 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

 $x_{ce1} = 50.000$ in

Dist center of baseplate to right edge foundation

 $x_{ce2} = 50.000$ in

Dist center of baseplate to bot edge foundation

 $y_{ce1} = 50.000$ in

Dist center of baseplate to top edge foundation

 $y_{ce2} = 50.000$ in

Holding down bolt and anchor plate details

Total number of bolts

 $N_{bolt} = 4$

Bolt diameter

 $d_o = 0.750$ in

Bolt spacing

$$S_{\text{bolt}} = 19.000 \text{ in}$$

Edge distance

$$e_1 = 1.500 \text{ in}$$

Minimum tensile strength of steel

$$F_y = 36 \text{ ksi}$$

Compressive strength of concrete

$$f'_c = 4 \text{ ksi}$$

Strength reduction factors

Compression

$$\phi_c = 0.65$$

Flexure

$$\phi_b = 0.90$$

Weld shear

$$\phi_v = 0.75$$

Plate cantilever dimensions

Area of base plate

$$A_1 = B \times N = 484.000 \text{ in}^2$$

Maximum area of supporting surface

$$A_2 = (N + 2 \times l_{\text{min}}) \times (B + 2 \times l_{\text{min}}) = 10000.000 \text{ in}^2$$

Nominal strength of concrete under base plate

$$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{A_2 / A_1}, 2) = 3291.2 \text{ kips}$$

Plate bending coefficient X

$$X = (4 \times d \times b_f) / (d + b_f)^2 \times \min(1.0, P_u / (P_p \times \phi_c)) = 0.12$$

Plate bending coefficient λ

$$\lambda_1 = \min(1, 2 \times \sqrt{X} / (1 + \sqrt{(1 - X)})) = 0.35$$

Bending line cantilever distance m

$$m = (N - 0.95 \times d) / 2 = 4.967 \text{ in}$$

Bending line cantilever distance n

$$n = (B - 0.8 \times b_f) / 2 = 6.120 \text{ in}$$

Yield line theory cantilever distance n'

$$n' = \sqrt{(d \times b_f) / 4} = 3.112 \text{ in}$$

Maximum bending line cantilever

$$l = \max(m, n, \lambda_1 \times n') = 6.120 \text{ in}$$

Check eccentricity

Maximum bearing stress

$$f_{p,\text{max}} = 0.85 \times f'_c \times \phi_c \times \min(\sqrt{A_2 / A_1}, 2) = 4.42 \text{ ksi}$$

Maximum bearing pressure

$$q_{\text{max}} = f_{p,\text{max}} \times B = 97.24 \text{ kips/in}$$

Critical eccentricity

$$e_{\text{crit}} = N / 2 - P_u / (2 \times q_{\text{max}}) = 9.715 \text{ in}$$

e > e_{crit} so loads cannot be resisted by bearing alone. Therefore consider as a large moment

Plate dimensions adequate as $(f + N/2)^2 \geq (2 \times P_u \times (e + f)) / q_{\text{max}}$ and a real solution for bearing length exists

Bearing length - quadratic solution 1

$$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f)) / q_{\text{max}})} = 38.215 \text{ in}$$

Bearing length - quadratic solution 2

$$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f)) / q_{\text{max}})} = 2.785 \text{ in}$$

Bearing length

$$Y = \min(Y_1, Y_2) = 2.785 \text{ in}$$

Tension force in bolts

$$T_u = q_{\text{max}} \times Y - P_u = 20.8 \text{ kips}$$

Max tension in single bolt

$$T_{\text{rod}} = T_u / (N_{\text{bolt}} / 2) = 10.4 \text{ kips}$$

Base plate yielding limit at bearing interface

Required plate thickness

$$t_{p,\text{req}} = \sqrt{((4 \times f_{p,\text{max}} \times Y \times (l - Y/2)) / (\phi_b \times F_y))} = 2.680 \text{ in}$$

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines

$$x = \text{abs}(l - e_1) = 4.620 \text{ in}$$

Plate thickness required

$$t_{p,\text{req}} = 2.11 \times \sqrt{(T_u \times x) / (B \times F_y)} = 0.736 \text{ in}$$

PASS - Thickness of plate exceeds required thickness

Frictional shear resistance

Steel / concrete friction coefficient

$$\mu = 0.4$$

Frictional shear resistance

$$\phi V_n = \min(\phi_v \times \mu \times q_{\text{max}} \times Y, \phi_v \times 0.2 \times f'_c \times Y \times B, \phi_v \times 800 \text{ psi} \times Y \times B) = 36.77 \text{ kips}$$

PASS - Frictional shear resistance exceeds applied shear

Flange weld

Flange weld leg length

$$t_{fw} = 0.2500 \text{ in}$$

Tension capacity of flange

$$P_{tf} = b_f \times t_f \times F_y = 395.3 \text{ kips}$$

Force in tension flange

$$F_{tf} = M_u / (d - t_f) - P_u \times (b_f \times t_f) / A_{col} = 139.9 \text{ kips}$$

Critical force in flange

$$F_f = \min(P_{tf}, \max(F_{tf}, 0 \text{ kips})) = 139.9 \text{ kips}$$

Flange weld force per in

$$R_{wf} = F_f / (2 \times b_f - t_w) = 5.9 \text{ kips/in}$$

Electrode classification number

$$F_{EXX} = 70.0 \text{ ksi}$$

Nominal weld stress

$$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90 \text{ deg}))^{1.5}) = 47.2500 \text{ ksi}$$

Design strength of weld per in

$$R_{nf} = F_w \times t_{fw} / \sqrt{2} = 10.4 \text{ kips/in}$$

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

Longitudinal web weld

Web weld leg length

$$t_{ww} = 0.2500 \text{ in}$$

Web weld force per in

$$R_{wl} = F_v / (2 \times (d - 2 \times t_f)) = 0.9174 \text{ kips/in}$$

Electrode classification number

$$F_{EXX} = 70.0 \text{ ksi}$$

Nominal weld stress

$$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90 \text{ deg}))^{1.5}) = 47.2500 \text{ ksi}$$

Design strength of weld per in

$$R_{nl} = F_w \times t_{ww} / \sqrt{2} = 8.4 \text{ kips/in}$$

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

ANCHOR BOLT DESIGN (ACI318-08)

TEDDS calculation version 2.0.00

Anchor bolt geometry

Type of anchor bolt

Cast-in headed end bolt anchor

Diameter of anchor bolt

$$d_a = 0.75 \text{ in}$$

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

$$S_{boltx} = 19 \text{ in}$$

Spacing of bolts in y direction

$$S_{bolty} = 19 \text{ in}$$

Number of threads per inch

$$n_t = 10$$

Effective cross-sectional area of anchor

$$A_{se} = \pi / 4 \times (d_a - 0.9743 \text{ in} / n_t)^2 = 0.334 \text{ in}^2$$

Embedded depth of each anchor bolt

$$h_{ef} = 18 \text{ in}$$

Material details

Minimum yield strength of steel

$$f_{ya} = 105 \text{ ksi}$$

Minimum tensile strength of steel

$$f_{uta} = \min(1.9 \times f_{ya}, 125 \text{ ksi}) = 125 \text{ ksi}$$

Compressive strength of concrete

$$f'_c = 4 \text{ ksi}$$

Concrete modification factor

$$\lambda = 1.00$$

Strength reduction factors

Strength reduction factor – tension of steel element $\phi_{t,s} = 0.75$

Strength reduction factor – shear of steel element $\phi_{v,s} = 0.70$

Strength reduction factor – concrete tension $\phi_{t,c} = 0.65$

Strength reduction factor – concrete shear $\phi_{v,c} = 0.70$

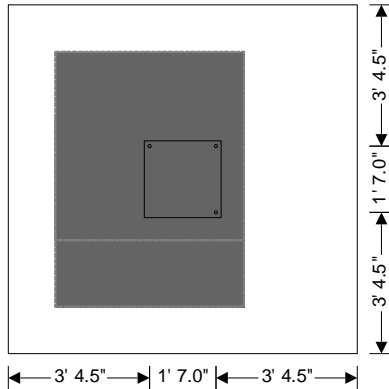
Steel strength of anchor in tension (D.5.1)

Nominal strength of anchor in tension

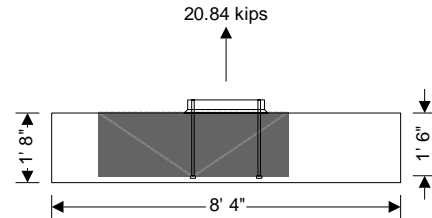
$$N_{sa} = A_{se} \times f_{uta} = \mathbf{41.81 \text{ kips}}$$

Steel strength of anchor in tension

$$\phi N_{sa} = \phi_{t,s} \times N_{sa} = \mathbf{31.36 \text{ kips}}$$

PASS - Steel strength of anchor exceeds max tension in single bolt
Check concrete breakout strength of anchor bolt in tension (D.5.2)


Plan on foundation



Section A-A

Concrete breakout - tension

Strength reduction factor

$$\phi_t = \mathbf{0.65}$$

Coeff for basic breakout strength in tension

$$k_c = \mathbf{24}$$

Breakout strength for single anchor in tension

$$N_b = 16 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times h_{ef}^{5/3} \times 1 \text{ in}^{1/3} = \mathbf{125.10 \text{ kips}}$$

Projected area for groups of anchors

$$A_{Nc} = \mathbf{3942 \text{ in}^2}$$

Projected area of a single anchor

$$A_{Nco} = 9 \times h_{ef}^2 = \mathbf{2916 \text{ in}^2}$$

Min dist center of anchor to edge of concrete

$$c_{a,min} = \mathbf{40.5 \text{ in}}$$

Mod factor for groups loaded eccentrically

$$\psi_{ec,N} = \min(1 / (1 + ((2 \times e'_N) / (3 \times h_{ef}))), 1) = \mathbf{1.000}$$

Modification factor for edge effects

$$\psi_{ed,N} = 1.0 = \mathbf{1.000}$$

Modification factor for no cracking at service loads

$$\psi_{c,N} = \mathbf{1.250}$$

Modification factor for uncracked concrete

$$\psi_{cp,N} = \mathbf{1.000}$$

Nominal concrete breakout strength

$$N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b = \mathbf{211.40 \text{ kips}}$$

Concrete breakout strength

$$\phi N_{cbg} = \phi_{t,c} \times N_{cbg} = \mathbf{137.41 \text{ kips}}$$

PASS - Breakout strength exceeds tension in bolts
Pullout strength (D.5.3)

Net bearing area of the head of anchor

$$A_{brg} = \mathbf{1.5 \text{ in}^2}$$

Mod factor for no cracking at service loads

$$\psi_{c,P} = \mathbf{1.4000}$$

Pullout strength for single anchor

$$N_p = 8 \times A_{brg} \times f'_c = \mathbf{48.00 \text{ kips}}$$

Nominal pullout strength of single anchor

$$N_{pn} = \psi_{c,P} \times N_p = \mathbf{67.20 \text{ kips}}$$

Pullout strength of single anchor

$$\phi N_{pn} = \phi_{t,c} \times N_{pn} = \mathbf{43.68 \text{ kips}}$$

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt



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Side face blowout strength (D.5.4)

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