Beam-Column Base Plate Design—LRFD Method

RICHARD M. DRAKE and SHARON J. ELKIN

INTRODUCTION

It is common design practice to design a building or structure beam-column with a moment-resisting or fixed base. Therefore the base plate and anchor rods must be capable of transferring shear loads, axial loads, and bending moments to the supporting foundation.

Typically, these beam-column base plates have been designed and/or analyzed by using service loads[1] or by approximating the stress relationship assuming the compression bearing location.[2] The authors present another approach, using factored loads directly in a method consistent with the equations of static equilibrium and the LRFD Specification.[3]

The moment-resisting base plate must have design strengths in excess of the required strengths, flexural (M_u), axial (P_u), and shear (V_u) for all load combinations.

A typical beam-column base plate geometry is shown in Figure 1, which is consistent with that shown on page 11-61 of the LRFD Manual.[4]

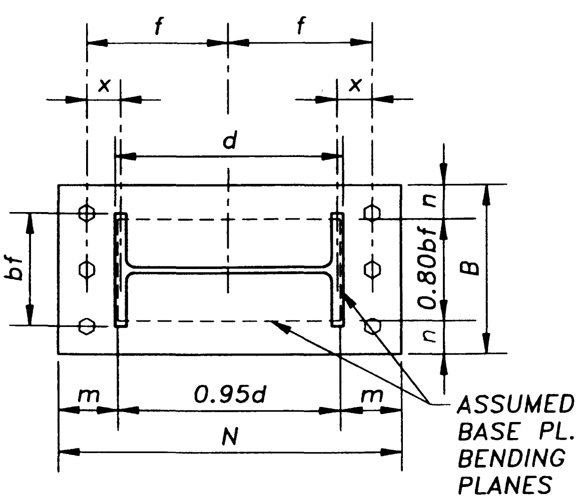

Fig. 1. Base Plate Design Variables

Richard M. Drake is Principal Structural Engineer, Fluor Daniel, Irvine, CA.

Sharon J. Elkin is Structural Engineer, Fluor Daniel, Irvine, CA.

where:

B = base plate width perpendicular to moment direction, in.
N = base plate length parallel to moment direction, in.
b_f = column flange width, in.
d = overall column depth, in.
f = anchor rod distance from column and base plate centerline parallel to moment direction, in.
m = base plate bearing interface cantilever direction parallel to moment direction, in.

$$m = \frac{N - 0.95d}{2} \tag{1}$$

n = base plate bearing interface cantilever perpendicular to moment direction, in.

$$n = \frac{B - 0.80b_f}{2} \tag{2}$$

x = base plate tension interface cantilever parallel to moment direction, in.

$$x = f - \frac{d}{2} + \frac{t_f}{2} \tag{3}$$

t_f = column flange thickness, in.

The progression of beam-column loadings, in order of increasing moments, is presented in four load cases.

Case A is a load case with axial compression and shear, without bending moment. This case results in a full length uniform pressure distribution between the base plate and the supporting concrete. This case is summarized in the LRFD Manual[4] beginning on page 11-54 and is summarized herein for completeness.

Case B evolves from Case A by the addition of a small bending moment. The moment changes the full length uniform pressure distribution to a partial length uniform pressure distribution, but is not large enough to cause separation between the base plate and the supporting concrete.

Case C evolves from Case B by the addition of a specific bending moment such that the uniform pressure distribution is the smallest possible length without separation between the base plate and the supporting concrete. This corresponds to the common elastic limit where any additional moment would initiate separation between the base plate and the supporting concrete.

Case D evolves from Case C by the addition of sufficient bending moment to require anchor rods to prevent separation between the base plate and the supporting concrete. This is a common situation for fixed base plates in structural office practice. That is, a rigid frame with a fixed base plate will usually attract enough bending moment to require anchor rods to prevent uplift of the base plate from the supporting concrete.

CASE A: NO MOMENT—NO UPLIFT

If there is no bending moment or axial tension at the base of a beam-column, the anchor rods resist shear loads but are not required to prevent uplift or separation of the base plate from the foundation. Case A, a beam-column with no moment or uplift at the base plate elevation, is shown in Figure 2.

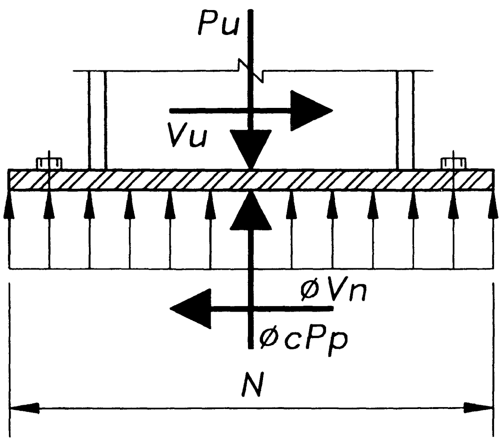

Fig. 2. No Moment - No Uplift

$$M_u = 0$$
$$P_u > 0$$

CASE B: SMALL MOMENT WITHOUT UPLIFT

If the magnitude of the bending moment is small relative to the magnitude of the axial load, the column anchor rods are not required to restrain uplift or separation of the base plate from the foundation. In service, they only resist shear. They are also necessary for the stability of the structure during construction.

AISC[5] addresses three different variations of the elastic method when using an ultimate strength approach for the design of beam-column base plates subjected to bending moment.

1. Assume that the resultant compressive bearing stress is directly under the column flange.
2. Assume a linear strain distribution such that the anchor rod strain is dependent on the bearing area strain.
3. Assume independent strain distribution.

All three methods summarized by AISC[5] assume a linear triangular distribution of the resultant compressive bearing stress. This implies that the beam-column base plate has no additional capacity after the extreme fiber reaches the concrete bearing limit state. The authors propose that a uniform distribution of the resultant compressive bearing stress is more appropriate when utilizing LRFD.

Case B, a beam-column with a small moment and no uplift at the base plate elevation, is shown in Figure 3. The moment M_u is expressed as P_u located at some eccentricity (e) from the beam-column neutral axis.

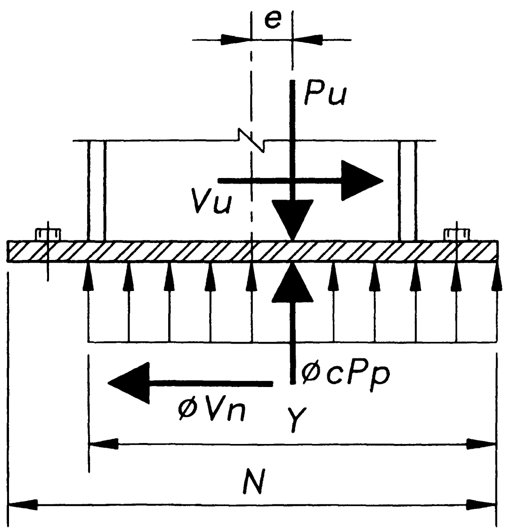

Fig. 3. Small Moment Without Uplift

$$e = \frac{M_u}{P_u} \tag{4}$$

$$0 < M_u < \frac{P_u N}{6}$$

$$0 < e < \frac{N}{6}$$

$$Y = N - 2e$$

$$e = \frac{N - Y}{2} \tag{5}$$

where:

Y = bearing length, in.

CASE C: MAXIMUM MOMENT WITHOUT UPLIFT

The maximum moment without base plate uplift is assumed to occur when the concrete bearing limit state is reached over a bearing area concentric with the applied load at its maximum eccentricity. If the eccentricity exceeds $\frac{N}{6}$, the tendency for uplift of the plate is assumed to occur. This assumes a linear pressure distribution in accordance with elastic theory and no tension capacity between the base plate and supporting concrete surfaces. Case C, a beam-column with the maximum moment without uplift at the base plate elevation, is shown in Figure 4.

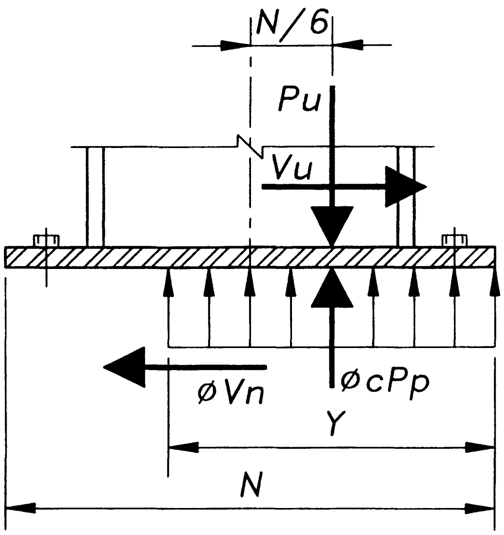

Fig. 4. Maximum Moment Without Uplift

$$e = \frac{M_u}{P_u} \tag{4}$$

$$0 < M_u = \frac{P_u N}{6}$$

$$e = \frac{N}{6}$$

$$Y = N - 2e = N - 2\left(\frac{N}{6}\right)$$

$$Y = \frac{2}{3}N \tag{6}$$

CASE D: MOMENT WITH UPLIFT

When the moment at the beam-column base plate exceeds $\frac{N}{6}$, anchor rods are designed to resist uplift as well as shear. Case D, a beam-column with sufficient moment to cause uplift at the base plate elevation, is shown in Figure 5. This is the most common case in design practice, especially for rigid frames designed to resist lateral earthquake or wind loadings on the building or structure.

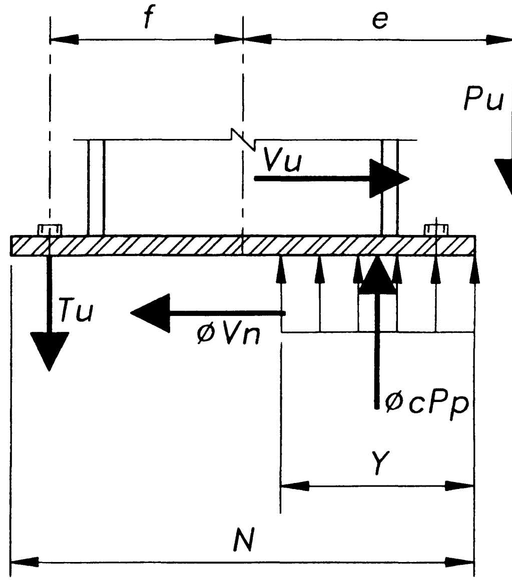

Fig. 5. Moment With Uplift

$$e = \frac{M_u}{P_u} \tag{4}$$

$$0 < \frac{P_u N}{6} < M_u$$

$$\frac{N}{6} < e \tag{7}$$

CONCRETE BEARING LIMIT STATE

To satisfy static equilibrium at the concrete bearing limit state, the centroid of the concrete bearing reaction (P_p) must be aligned with the line-of-action of the applied axial load.

LRFD Specification Requirements

The LRFD Specification[3] defines the concrete bearing limit state in Section J9.

$$P_u \leq \phi_c P_p \tag{8}$$

On the full area of a concrete support:

$$P_p = 0.85\, f_c' A_1 \qquad \text{(LRFD J9-1)}$$

On less than the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \qquad \text{(LRFD J9-2)}$$

$$\sqrt{\frac{A_2}{A_1}} \le 2$$

where:

ϕ_c = compression resistance factor = 0.60
f_c' = specified concrete compressive strength, ksi
A_1 = area of steel concentrically bearing on a concrete support, in.^{2}
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

Practical Design Procedure—Required Area

Select base plate dimensions such that:

$$P_u \le \phi_c P_p \qquad (8)$$

And noting that:

$$M_u = P_u e \qquad (9)$$

For convenience, define a new variable, q, the concrete bearing strength per unit width (K/in).

$$q = \phi_c 0.85 f_c' B \sqrt{\frac{A_2}{A_1}} \le \phi_c 0.85 f_c' B(2)$$

$$q = (0.60)(0.85) f_c' B \sqrt{\frac{A_2}{A_1}} \le (0.60)(0.85) f_c' B(2)$$

$$q = 0.51 f_c' B \sqrt{\frac{A_2}{A_1}} \le 1.02 f_c' B \qquad (10)$$

For most column base plates bearing directly on a concrete foundation, the concrete dimension is much greater than the base plate dimension, and it is reasonable to assume that the ratio $\sqrt{\frac{A_2}{A_1}} \ge 2$. For most column base plates bearing on grout or a concrete pier, the concrete (grout) dimension is equal to the base plate dimension, and it is reasonable to conservatively take the ratio $\sqrt{\frac{A_2}{A_1}} = 1$.

Case A: No Moment - No Uplift

$$A_1 = BN$$

$$P_u \le (0.60)(0.85) f_c' BN \sqrt{\frac{A_2}{A_1}} \le (0.60)(0.85) f_c' BN(2)$$

$$P_u \le qN \qquad (11)$$

Case B: Small Moment Without Uplift

$$A_1 = BY$$

$$Y = (N - 2e)$$

$$P_u \le (0.60)(0.85) f_c' BY \sqrt{\frac{A_2}{A_1}} \le (0.60)(0.85) f_c' BY(2)$$

$$P_u \le qY$$

$$P_u \le q(N - 2e) \qquad (12)$$

Note that equation 12 is not a closed form solution because;

- q is a function of A_1,
- A_1 is a function of y,
- y is a function of e, and
- e is a function of P_u.

However, if e is defined as some fixed distance or as some percentage of N, the corresponding maximum values of P_u and M_u can be determined directly.

Case C: Maximum Moment Without Uplift

As previously stated, Case C is the situation where uplift is imminent and $e = \frac{N}{6}$.

$$A_1 = BY$$

$$Y = \frac{2}{3}N \qquad (6)$$

$$P_u \le (0.60)(0.85) f_c' BY \sqrt{\frac{A_2}{BY}} \le (0.60)(0.85) f_c' BY(2)$$

$$P_u \le 0.51 f_c' B\left(\frac{2}{3}N\right) \sqrt{\frac{A_2}{B\left(\frac{2}{3}N\right)}} \le 1.02 f_c' B\left(\frac{2}{3}N\right)$$

$$P_u \le 0.667 qN \qquad (13)$$

$$M_u = P_u(e) = P_u\left(\frac{N}{6}\right)$$

$$M_u \le 0.111 qN^2 \qquad (14)$$

Case D: Moment with Uplift

Given the following:

P_u, M_u, ϕ_c, f_c', B, f {inches & kips}

$$\phi_c P_p = \phi_c 0.85 f_c' BY \sqrt{\frac{A_2}{A_1}} = qY \qquad (15)$$

$$e = \frac{M_u}{P_u} \qquad (4)$$

Two equations will be needed to solve for the two unknowns, the required tensile strength of the anchor rods, T_u, and bearing length, Y.

To maintain static equilibrium, the summation of vertical force must equal zero:

$$\Sigma F_{\text{vertical}} = 0$$

$$T_u + P_u - \phi_c P_p = 0$$

$$T_u = qY - P_u \tag{16}$$

To maintain static equilibrium, the summation of moments taken about the force T_u must equal zero:

$$\phi_c P_p\left(\frac{N}{2} - \frac{Y}{2} + f\right) - P_u(e + f) = 0$$

$$qY\left(\frac{N}{2} - \frac{Y}{2} + f\right) - P_u(e + f) = 0 \tag{17}$$

$$\frac{qYN}{2} - \frac{qY^2}{2} + qYf - P_u(e + f) = 0$$

$$\left(\frac{q}{2}\right)Y^2 - q\left(f + \frac{N}{2}\right)Y + P_u(e + f) = 0 \tag{18}$$

This is in the form of a classic quadratic equation, with unknown Y.

$$aY^2 + bY + c = 0 \tag{19}$$

$$Y = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$Y = \frac{q\left(f + \frac{N}{2}\right) \pm \sqrt{\left[-q\left(f + \frac{N}{2}\right)\right]^2 - 4\left(\frac{q}{2}\right)[P_u(f + e)]}}{2\left(\frac{q}{2}\right)}$$

$$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left[-\left(f + \frac{N}{2}\right)\right]^2 - \frac{2P_u(f + e)}{q}} \tag{20}$$

To determine the other unknown, T_u, substitute the value for Y into the equation:

$$T_u = qY - P_u \tag{16}$$

As a check, back substitute the value for Y into the equation:

$$qY\left(\frac{N}{2} - \frac{Y}{2} + f\right) - P_u(e + f) = 0 \tag{17}$$

ANCHOR ROD SHEAR AND TENSION LIMIT STATES

LRFD Specification Requirements

The LRFD Specification[3] defines the anchor rod (bolts) shear and tension limit states in Sections J3.6 and J3.7, and Tables J3.2 and J3.5.

$$V_{ub} \le \phi F_v A_b \tag{21}$$

$$T_{ub} \le \phi F_t A_b \tag{22}$$

For ASTM A307 bolts:

$$F_t = 59 - 1.9f_v \le 45 \qquad \text{(Table J3.5)}$$

For ASTM A325 bolts, threads excluded from the shear plane:

$$F_t = 117 - 1.5f_v \le 90 \qquad \text{(Table J3.5)}$$

where:

V_{ub} = required anchor rod shear strength, kips
ϕ = anchor rod resistance factor = 0.75
F_v = nominal shear strength, ksi
A_b = anchor rod nominal (gross) area, in.^{2}
T_{ub} = required anchor rod tensile strength, kips
F_t = nominal tensile strength, ksi
f_v = anchor rod shear stress, ksi

$$f_v = \frac{V_{ub}}{A_b} \tag{23}$$

For A307 bolts:

$$F_v = 24 \text{ ksi} \qquad \text{(Table J3.2)}$$

For A325 bolts when threads are excluded from the shear plane:

$$F_v = 60 \text{ ksi} \qquad \text{(Table J3.2)}$$

Required Strength

The shear stress (f_v) is calculated considering the required shear strength of the column base.

$$f_v = \frac{V_{ub}}{\#_v A_b} \tag{24}$$

where:

$\#_v$ = number of rods sharing shear load, unitless

Note that all the base plate anchor rods are considered effective in sharing the shear load.

Practical Design Procedure—Rod Sizes

$$V_{ub} = \frac{V_u}{\#_v} \le 0.75F_v A_b \tag{25}$$

$$F_t = 59 - 1.9\left(\frac{V_{ub}}{A_b}\right) \leq 45 \tag{26}$$

$$T_{ub} = \frac{T_u}{\#_t} \leq 0.75F_tA_b \tag{27}$$

where:

$\#_t$ = number of rods sharing tension load, unitless

Note that all of the base plate anchor rods are not considered effective in sharing the tension load. For most base plate designs, only half of the anchor rods are required to resist tension for a given load combination.

The embedment, edge distances, and overlapping shear cones of the anchor rods into the concrete must be checked to assure that the design tensile strength also exceeds the required tensile strength. This check should be in accordance with the appropriate concrete design specification, and is beyond the scope of this paper.[3,6]

It should be noted that base plate holes are often oversized with respect to the anchor rods. In this case, some "slippage" may be necessary before the anchor rod shear limit state is reached. For large shear loads, the designer may choose to investigate alternate shear transfer limit states involving pretensioned bolts,[7] friction and/or shear lugs.

BASE PLATE FLEXURAL YIELDING LIMIT STATE

The entire base plate cross-section can reach the specified yield stress (F_y).

LRFD Specification Requirements

The LRFD Specification[3] defines the flexural yielding limit state in Section F1.

$$M_{pl} \leq \phi_b M_n \tag{28}$$

$$M_n = M_p \qquad \text{(LRFD F1-1)}$$

where:

M_{pl} = required base plate flexural strength, in-K
ϕ_b = flexural resistance factor = 0.90
M_n = nominal flexural strength, in-K
M_p = plastic bending moment, in-K

Required Strength—Bearing Interface

The bearing pressure between the concrete and the base plate will cause bending in the base plate for the cantilever distances m and n. The bearing stress, f_p (ksi), is calculated considering the required axial and flexural strength of the column base, P_u and M_u respectively.

On section parallel to column flanges:

$$M_{pl} = f_p\left(\frac{m^2}{2}\right) \tag{29}$$

On section parallel to column web:

$$M_{pl} = f_p\left(\frac{n^2}{2}\right) \tag{30}$$

where:

f_p = concrete bearing stress, ksi

The bearing pressure may cause bending in the base plate in the area between the flanges, especially for lightly loaded columns. Yield line theory[8,9] is used to analyze this consideration.

$$n' = \frac{\sqrt{db_f}}{4} \tag{31}$$

$$M_{pl} = f_p\frac{(n')^2}{2} \tag{32}$$

Let c = the larger of m, n, and n':

$$M_{pl} = f_p\left(\frac{c^2}{2}\right) \tag{33}$$

where:

n' = yield line theory cantilever distance from column web or column flange, in.
c = largest base plate cantilever, in.

Note that for most base plate geometries, the cantilever dimension (n) is very small and "corner bending" of the base plate is neglected. When the dimension is large to accommodate more anchor rods or more bearing surface, corner bending plate moments should be considered and used in the base plate thickness calculations.

Required Strength—Tension Interface

The tension on the anchor rods will cause bending in the base plate for the cantilever distance x.

For a unit width of base plate:

$$M_{pl} = \frac{T_u x}{B} \tag{34}$$

Nominal Strength

For a unit width of base plate:

$$M_n = M_p = \left(\frac{t_p^2}{4}\right)F_y \tag{35}$$

Practical Design Procedure—Bearing Interface Base Plate Thickness

Setting the design strength equal to the nominal strength and solving for the required plate thickness (t_p):

$$M_{pl} \leq \phi_b M_n \qquad (28)$$

$$M_n = M_p \qquad \text{(LRFD F1-1)}$$

$$f_p\left(\frac{c^2}{2}\right) \leq 0.90\left(\frac{t_p^2}{4}\right)F_y$$

$$t_{p(req)} \geq 1.49c\sqrt{\frac{f_p}{F_y}} \qquad (36)$$

Practical Design Procedure—Tension Interface Base Plate Thickness

Setting the design strength equal to the nominal strength and solving for the required plate thickness:

$$M_{pl} \leq \phi_b M_p \qquad (28)$$

$$\frac{T_u x}{B} \leq 0.90\left(\frac{t_p^2}{4}\right)F_y$$

$$t_{p(req)} \geq 2.11\sqrt{\frac{T_u x}{BF_y}} \qquad (37)$$

Case A: No Moment—No Uplift

$$f_p = \frac{P_u}{BN} \qquad (38)$$

$$t_{p(req)} \geq 1.49c\sqrt{\frac{P_u}{BNF_y}} \qquad (39)$$

Case B: Small Moment Without Uplift

$$f_p = \frac{P_u}{BY} = \frac{P_u}{B(N-2e)} \qquad (40)$$

$$t_{p(req)} \geq 1.49c\sqrt{\frac{P_u}{B(N-2e)F_y}} \qquad (41)$$

Case C: Maximum Moment Without Uplift

$$f_p = \frac{P_u}{BY} = \frac{P_u}{B\left(\frac{2}{3}N\right)} = \frac{1.5P_u}{BN} \qquad (42)$$

$$t_{p(req)} \geq 1.49c\sqrt{\frac{1.5P_u}{BNF_y}} \qquad (43)$$

Case D: Moment with Uplift

$$f_p = \frac{P_u}{BY} \qquad (44)$$

For all cases:

$$t_{p(req)} \geq 2.11\sqrt{\frac{T_u x}{BF_y}} \qquad (45)$$

If $Y > m$:

$$t_{p(req)} \geq 1.49c\sqrt{\frac{P_u}{BYF_y}} \qquad (46)$$

If $Y < m$:

$$t_{p(req)} \geq 2.11\sqrt{\frac{P_u\left(m - \frac{Y}{2}\right)}{BF_y}} \qquad (47)$$

DESIGN EXAMPLE 1

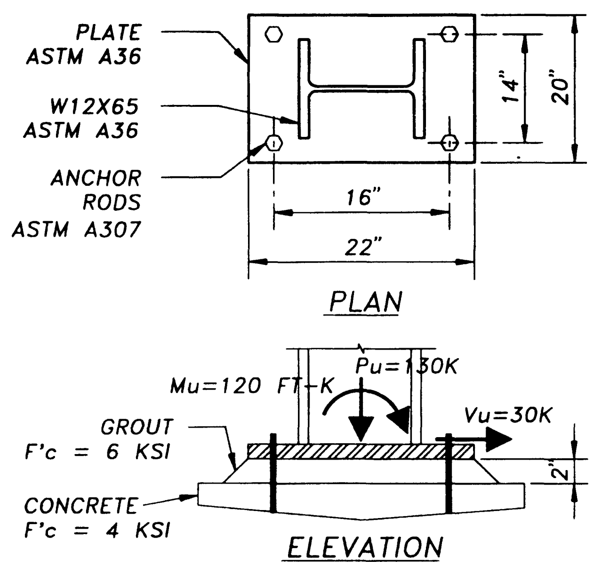

Fig. 6. Design Example 1

Required:

a) Design anchor rods
b) Determine base plate thickness

Solution:

1. Dimensions:

$$m = \frac{22.0\text{ in.} - 0.95(12.12\text{ in.})}{2} = 5.24\text{ in.} \qquad (1)$$

$$x = \frac{16.0\text{ in.}}{2} - \frac{12.12\text{ in.}}{2} + \frac{0.605\text{ in.}}{2} = 2.24\text{ in.} \qquad (3)$$

2. Eccentricity:

$$e = \frac{120 \text{ ft-K}(12 \text{ in./ft})}{130\text{K}} = 11.08 \text{ in.} \qquad (4)$$

$$\frac{N}{6} = \frac{22.0 \text{ in.}}{6} = 3.67 \text{ in.} < 11.08 \text{ in.} = e, \ \textit{Case D}(7)$$

3. Concrete bearing:
Assume the bearing on grout area will govern.

$$q = (0.51)(6 \text{ ksi})(20.0 \text{ in.})\sqrt{1} = 61.2 \text{ K/in.} \qquad (10)$$

$$f + \frac{N}{2} = \frac{16.0 \text{ in.}}{2} + \frac{22.0 \text{ in.}}{2} = 19.0 \text{ in.}$$

$$f + e = \frac{16.0 \text{ in.}}{2} + 11.08 \text{ in.} = 19.08 \text{ in.}$$

$$Y = 19.0 \pm \sqrt{(19.0)^2 - \frac{2(130)(19.08)}{61.2}} \qquad (20)$$

$$= 19.0 \pm 16.73 = 2.27 \text{ in.}$$

$$T_u = 61.2 \text{ K/in.}(2.27 \text{ in.}) - 130 \text{ K} = 8.92 \text{ K} \qquad (16)$$

4. Anchor rod shear and tension:
Check $4 - \frac{3}{4}$in. dia. anchor rods

$$V_{ub} = \frac{30.0 \text{ K}}{4} = 7.50 \text{ K} \qquad (25)$$

$$\phi F_v A_b = 0.75(24 \text{ ksi})(0.4418 \text{ in.}^2)$$

$$= 7.96 \text{ K} > 7.50 \text{ K} = V_{ub} \quad \textbf{o.k.}$$

$$F_t = 59 - 1.9\left(\frac{7.50 \text{ K}}{0.4418 \text{ in.}^2}\right) = 26.7 \text{ ksi} \qquad (26)$$

$$T_{ub} = \frac{8.92 \text{ K}}{2} = 4.46 \text{ K} \qquad (27)$$

$$\phi F_t A_b = 0.75(26.7 \text{ ksi})(0.4418 \text{ in.}^2)$$

$$= 8.85 > 4.46 \text{ K} = T_{ub} \quad \textbf{o.k.}$$

Select: 4 - 3/4 in. Diameter Anchor Rods

5. Base plate flexural yielding:

$Y = 2.27$ in. < 5.24 in. $= m$, n and n' not applicable

$$t_{p(req)} = 2.11\sqrt{\frac{(8.92 \text{ K})(2.24 \text{ in.})}{(20.0 \text{ in.})(36 \text{ ksi})}} = 0.35 \text{ in.} \qquad (45)$$

$$t_{p(req)} = 2.11\sqrt{\frac{(130 \text{ K})\left(5.24 \text{ in.} - \frac{2.27 \text{ in.}}{2}\right)}{(20.0 \text{ in.})(36 \text{ ksi})}} \qquad (47)$$

$= 1.82$ in. *controls*

Select: Base Plate 2 × 20 × 1'-10

6. Check bearing on concrete below grout layer
The grout is 2 in. thick. Assume that the concrete extends at least 2 in. beyond grout in each direction.

$$q = (0.51)(4 \text{ ksi})(20.0 \text{ in.})\sqrt{\frac{(24 \text{ in.})(6.67 \text{ in.})}{(20 \text{ in.})(2.27 \text{ in.})}} \qquad (10)$$

$= 76.6$ K/in. > 61.2 K/in. used in design **o.k.**

DESIGN EXAMPLE 2

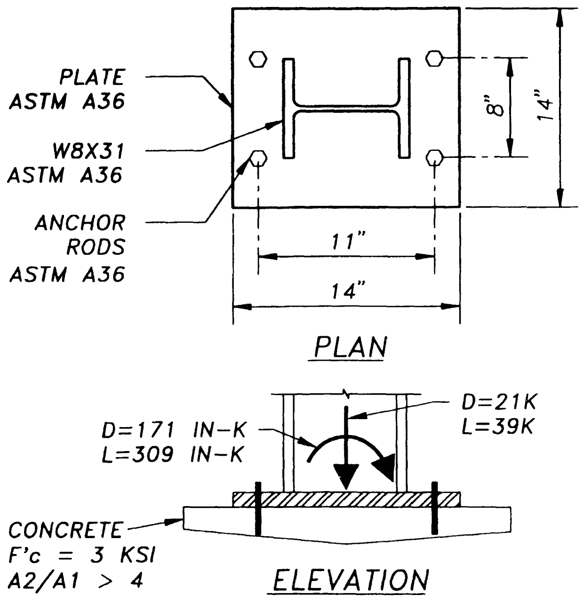

Fig. 7. Design Example 2

Required:

a) Determine required tensile strength
b) Determine base plate thickness

Solution:

Note that this problem is Example 16 from the AISC Column Base Plate Steel Design Guide Series.[5]

1. Required strength: (LRFD A4-2)

$$P_u = 1.2(21\text{K}) + 1.6(39\text{K}) = 87.6\text{K}$$

$$M_u = 1.2(171 \text{ in.-K}) + 1.6(309 \text{ in.-K}) = 700 \text{ in.-K}$$

2. Dimensions:

$$m = \frac{14.0 \text{ in.} - 0.95(7.995 \text{ in.})}{2} = 3.20 \text{ in.} \qquad (1)$$

$$x = \frac{11.0 \text{ in.}}{2} - \frac{7.995 \text{ in.}}{2} + \frac{0.435 \text{ in.}}{2} = 1.72 \qquad (3)$$

3. Eccentricity:

$$e = \frac{700 \text{ in.-K}}{87.6 \text{ K}} = 7.99 \text{ in.} \quad (4)$$

$$\frac{N}{6} = \frac{14.0 \text{ in.}}{6} = 2.33 \text{ in.} < 7.99 \text{ in.} = e, \ \textit{Case D} \ (7)$$

4. Concrete bearing:

$$q = (0.51)(3 \text{ ksi})(14 \text{ in.})\sqrt{4} = 42.8 \text{ K/in.} \quad (10)$$

$$f + \frac{N}{2} = \frac{11.0 \text{ in.}}{2} + \frac{14.0 \text{ in.}}{2} = 12.5 \text{ in.}$$

$$f + e = \frac{11.0 \text{ in.}}{2} + 7.99 \text{ in.} = 13.49 \text{ in.}$$

$$Y = 12.5 \pm \sqrt{(-12.5)^2 - \frac{2(87.6)(13.49)}{42.8}} \quad (20)$$

$$= 12.5 \pm 10.05 = 2.45 \text{ in.}$$

$$T_u = 42.8 \text{ K/in.}(2.45 \text{ in.}) - 87.6 \text{ K} = 17.3 \text{ K} \quad (16)$$

Required Tensile Strength = 17.3 K

5. Base plate flexural yielding:

$Y = 2.45$ in. < 3.20 in. $= m$, n and n' not applicable

$$t_{p(req)} = 2.11\sqrt{\frac{(17.3 \text{ K})(1.72 \text{ in.})}{(14.0 \text{ in.})(36 \text{ ksi})}} = 0.51 \text{ in.} \quad (45)$$

$$t_{p(req)} = 2.11\sqrt{\frac{(87.6 \text{ K})\left(3.20 \text{ in.} - \frac{2.45 \text{ in.}}{2}\right)}{(14.0 \text{ in.})(36 \text{ ksi})}} \quad (47)$$

$$= 1.24 \text{ in. } \textit{controls}$$

Select: Base Plate 1¼ × 14 × 1′-2

6. Comparison:

AISC[5] solution for this problem:

Required Anchor Rod Tensile Strength = 21.2 K

Select: Base Plate 1¼ × 14 × 1′-2

Length of triangular compression block = 5.1 in.

Author's solution for this problem:

Required Anchor Rod Tensile Strength = 17.3 K

Select: Base Plate 1¼ × 14 × 1′-2

Length of rectangular compression block = 2.45 in.

Remarks:

The authors' solution yields the identical base plate size and thickness. Required tensile strength for the design of the anchor rods is slightly smaller because the centroid of the compression reaction is a greater distance from the anchor rods.

SUMMARY AND CONCLUSIONS

A methodology has been presented that summarizes the design of beam-column base plates and anchor rods using factored loads directly in a manner consistent with the equations of static equilibrium and the LRFD Specification.[3] Two design examples have been presented. A direct comparison was made with a problem solved by another AISC method.

The step-by-step methodology presented will be beneficial in a structural design office, allowing the design practitioner to use the same factored loads for the design of the steel structure, base plate, and anchor rods. In addition the uniform "rectangular" pressure distribution will be easier to design and program than the linear "triangular" pressure distribution utilized in allowable stress design and other published LRFD formulations.[5]

REFERENCES

1. Blodgett, Omer W., *Design Of Welded Structures*, 1966.
2. Smith, J. C., *Structural Steel Design, LRFD Approach*, 2nd Edition, 1996.
3. American Institute of Steel Construction (AISC), "Load and Resistance Factor Design Specification for Structural Steel Buildings", December 1, 1993.
4. American Institute of Steel Construction (AISC), *Manual Of Steel Construction, Load & Resistance Factor Design*, 2nd Edition, Volume 2, 1994.
5. American Institute of Steel Construction (AISC), *Column Base Plates*, Steel Design Guide Series, 1990.
6. Shipp, J.G., and Haninger, E.R., "Design Of Headed Anchor Bolts," *Engineering Journal,* Vol 20, No. 2, (2nd Qtr.), pp 58-69, AISC, 1983.
7. American Society of Civil Engineers (ASCE), *Design Of Anchor Bolts In Petrochemical Facilities*, pp 4-3 to 4-8, 1997.
8. Thornton, W. A., "Design of Small Base plates for Wide-Flange Columns," *Engineering Journal*, Vol 27, No. 3, (3rd Qtr.), pp 108-110, AISC, 1990a.
9. Thornton, W. A., "Design of Small Base plates for Wide-Flange Columns - A Concatenation of Methods," *Engineering Journal*, Vol 27, No. 4, (4th Qtr.), pp 108-110, AISC, 1990b.

NOMENCLATURE

A_1 = area of steel concentrically bearing on a concrete support, in.^{2}

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}
A_b = anchor rod nominal (gross) area, in.^{2}
B = base plate width perpendicular to moment direction, in.
F_t = nominal tensile strength, ksi
F_v = nominal shear strength, ksi
F_y = specified minimum yield stress, ksi
M_n = nominal flexural strength, in.-K
M_p = plastic bending moment, in.-K
M_{pl} = required base plate flexural strength, in.-K
M_u = required flexural strength, in.-K
N = base plate length parallel to moment direction, in.
P_p = nominal bearing load on concrete, kips
P_u = required axial strength, kips
T_u = required tensile strength, kips
T_{ub} = required anchor rod tensile strength, kips
V_u = required shear strength, kips
V_{ub} = required anchor rod shear strength, kips
Y = bearing length, in.
b_f = column flange width, in.
c = largest base plate cantilever, in.
d = column overall depth, in.
e = axial eccentricity, in.
f = anchor rod distance from column and base plate centerline parallel to moment direction, in.
f'_c = specified concrete compressive strength, ksi
f_p = concrete bearing stress, ksi
f_v = anchor rod shear stress, ksi
m = base plate bearing interface cantilever parallel to moment direction, in.
n = base plate bearing interface cantilever perpendicular to moment direction, in.
n' = yield line theory cantilever distance from column web or column flange, in.
q = concrete (or grout) bearing strength per unit width, kips/in.
t_f = column flange thickness, in.
t_p = base plate thickness, in.
x = base plate tension interface cantilever parallel to moment direction, in.
ϕ = anchor rod resistance factor = 0.75
ϕ_b = flexural resistance factor = 0.90
ϕ_c = compression resistance factor = 0.60
$\#_t$ = number of rods sharing tension load, unitless
$\#_v$ = number of rods sharing shear load, unitless