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Steel Column and Base Plate Design Fundamentals

by

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1.0 Course Overview

This course covers procedures used in the analysis and design of steel structural columns. Content begins with general concepts based on mechanics of materials and builds into design concepts based on structural design code. Design examples will be presented using both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) methods. The course will focus on columns that are concentrically loaded, though moment resisting base plates will be briefly discussed.

The intent of this course is not to provide an exhaustive coverage of column design. It does, however, cover fundamental concepts used in common applications. The course covers design of the column, along with base plate design. Column cross-sections will be limited to wide flange sections and hollow structural sections because both are common for structural columns.

2.0 Basic Concepts of Column Analysis

2.1 Introduction to Columns

Axially loaded members can be in tension or compression. Tension members are designed simply by determining axial stress (force divided by area) and comparing that value to an allowable stress of the material. In the most basic definition, columns are axially loaded compression members. The compressive load can be concentric, where the load is applied along the column's centroidal axis, or it can be eccentric, where the load is applied away from the centroidal axis yet parallel to that axis.

When designing columns, the effects of instability must also be considered. As the column's length increases, there will be reduction in the load capacity. The slenderness ratio is a dimensionless term used to indicate the column's tendency to buckle and is expressed as the unbraced length (in) divided by the least radius of gyration (in).

Slenderness Ratio $$\frac{L}{r}$$ Equation 1

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Radius of gyration is a distance from the neutral axis where a theoretical point containing the section's concentrated area would be located to create the same moment of inertia.

Columns are usually subdivided into two groups: long and intermediate. A third category of short columns can be included, but buckling does not occur in short columns and compressive stress of force divided by cross-sectional area is used. Determination of the proper group is based on slenderness ratio.

2.1.1 Long Columns

For long columns, the Euler formula is used to predict the column's strength for elastic buckling (axial buckling stress remains below proportional limit). Euler's analysis was based on the differential equation of the elastic curve, which is only valid up to the proportional limit. Derivation of the formula is beyond the scope of this course, but the critical buckling load is computed with the Euler formula given by

$$P_{cr} = \frac{\pi^2 EI}{L^2}$$

Equation 2

where E is the material's modulus of elasticity (psi), I is the cross-section's moment of inertia about the axis of buckling (in⁴), and L is the column's effective length [see Section 2.2] (in) to give critical buckling load in pounds.

Equation 2 is usually written in a different form to include slenderness ratio. Using the substitution $I = Ar^2$ gives an expression for a critical stress.

$$\sigma_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 E}{\left(L/r\right)^2}$$

Equation 3

Because the Euler formula is based on stresses in the elastic region, a long column is theoretically defined as one in which stresses at the critical buckling load are less than the material's proportional limit. To illustrate, Equation 3 is plotted in Figure 1 using a modulus of

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elasticity of 29,000 ksi and a yield stress of 36 ksi. For values below the yield stress, Equation 3 is plotted as a solid black line. The line becomes a dashed red line for values above the yield stress. From this plot it can be determined that the long column equation (critical buckling) is theoretically only valid for slenderness ratio above approximately 90 for the material properties given. For slenderness ratios below that value, buckling becomes inelastic and Equation 3 is no longer valid.
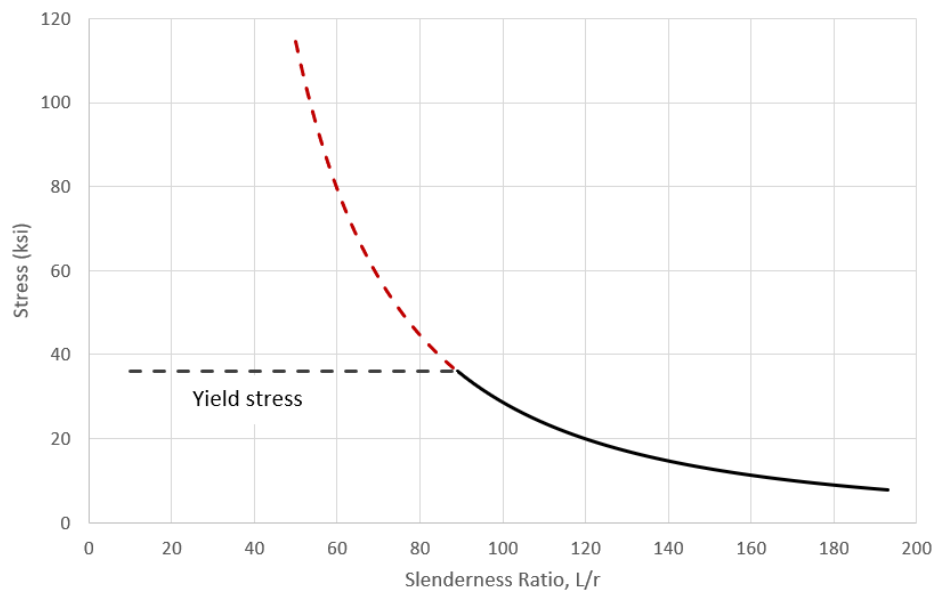

Figure 1   Critical stress vs. slenderness ratio ($E = 29,000\ ksi$, $\sigma_y = 36\ ksi$)

2.1.2   Intermediate Columns

For intermediate columns the member will fail by both buckling and yielding. Some of the fibers will reach yield stress, so their behavior is inelastic buckling. The Euler equation can be modified for intermediate columns using the tangent modulus of elasticity (E_t). The tangent modulus of elasticity is the slope of the stress-strain curve at the specific value of stress where buckling occurs. The modulus of elasticity is the slope in the elastic region, and the tangent

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modulus of elasticity is the slope of the stress-strain curve beyond the proportional limit. Using the tangent modulus of elasticity give

$$P_{cr} = \frac{\pi^2 E_t I}{L^2}$$
Equation 4

Use of the tangent modulus requires a stress-strain diagram for the material used, and it is preferred to have the diagram specifically for compression. Perhaps a more practical approach to determining allowable stress in intermediate columns is the use of Equation 5.

$$\sigma = \left(1 - \frac{(L/r)^2}{2C_c^2}\right)\sigma_y$$
Equation 5

The term C_c is the slenderness ratio value defining the transition from long column to intermediate column and is calculated as

$$C_c = \sqrt{\frac{2E\pi^2}{\sigma_y}}$$
Equation 6

As a common example for steel, using $E = 29{,}000\ ksi$ and $\sigma_y = 36\ ksi$, the value of C_c will be a slenderness ratio of 126. Note that this is more conservative than the theoretical value determined from Figure 1. A plot of allowable stress can then be developed as shown in Figure 2. The solid black line represents intermediate columns and is a plot of Equation 5 up to a slenderness ratio determined from Equation 6. The black dashed line represents long columns and is a plot of Equation 3 for values above C_c.

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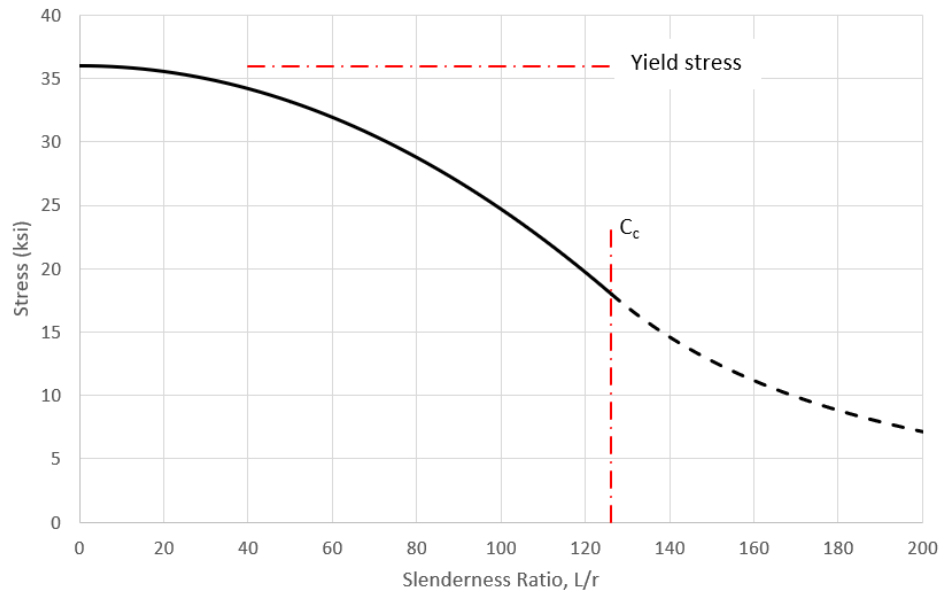*A SunCam online continuing education course*

Figure 2   Critical stress vs. slenderness ratio for intermediate columns and long
columns ($E = 29{,}000\ ksi,\ \sigma_y = 36\ ksi$)

2.2 Effective Length

Another very important factor for determining the buckling behavior of a column would be the
column's end conditions, or how the column is supported on each end. Consider the column of
length L illustrated in Figure 3 (a). Both ends are pinned connections, so they are free to rotate
but fixed in translation. Under loading, the column will tend to buckle to the exaggerated
deformed shape shown as the dashed line.

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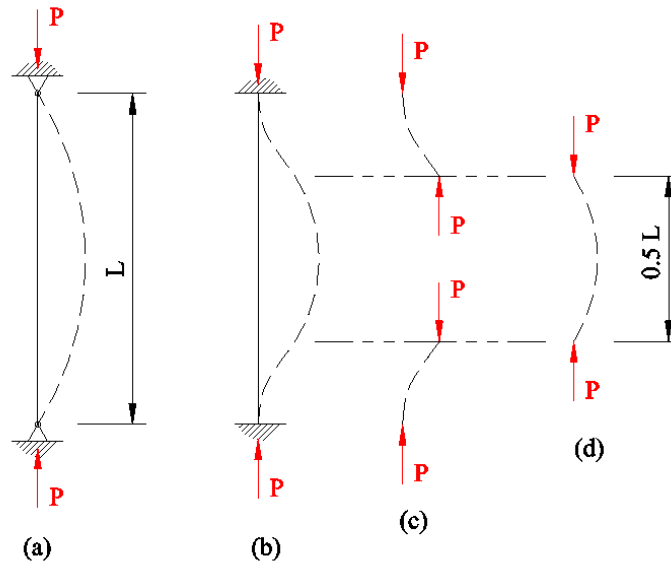*A SunCam online continuing education course*

Figure 3   Concept of effective length

Consider next the column shown in Figure 3 (b). The column length remains *L*, but the end conditions have changed to fixed ends (rotation is fixed, and translation is fixed). Under load, the column will tend to buckle in the shape shown as the dashed line. The buckled shape can be separated at the inflection points as shown in Figure 3 (c) and Figure 3 (d). The free body diagrams only show summation of forces in the vertical direction for clarity. The diagram of the buckled shape in Figure 3 (d) will be the same theoretical shape as that of the original pinned connections shown in Figure 3 (a) with half the length. Therefore, the column shown in Figure 3 (b) will have a theoretical effective length of 0.5*L* when compared to that shown in Figure 3 (a). Consequently, effective length is the distance between inflection points.

Design codes have recommended effective lengths to use for different end conditions, and those may differ from the theoretical effective lengths to be conservative. Recommended effective lengths are shown in Figure 4 for different column end conditions. The factor being multiplied by the unbraced length *L* is known as the effective length factor *K*.

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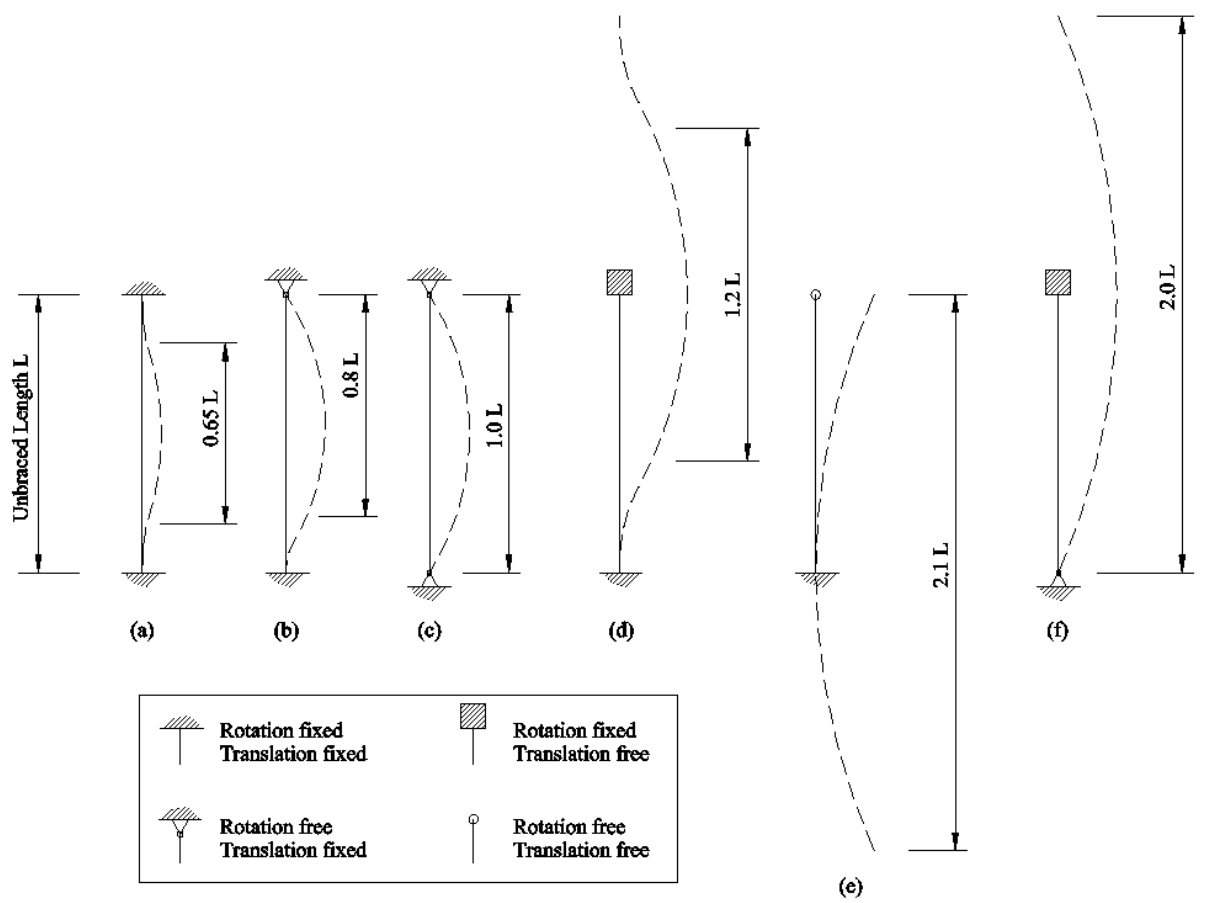
Figure 4   Effective length factors, *K*

Sidesway is when the top of the column is relatively free to move laterally with respect to the
bottom of the column. The first three conditions shown in Figure 4 (a, b, and c) are cases when
sidesway is prevented, and the remaining three (d, e, and f) are cases when sidesway occurs.
When sidesway is prevented, the effective length will be less than or equal to the unbraced
length. When sidesway occurs, the effective length will be greater than the unbraced length.

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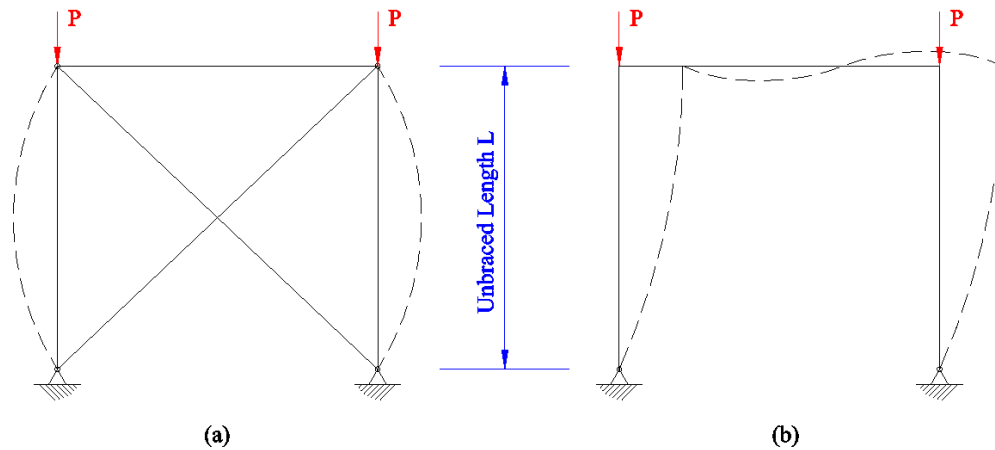*A SunCam online continuing education course*

Figure 5   (a) Sidesway prevented, (b) Sidesway occurs

Figure 5 (a) shows columns with X bracing, which is an application where sidesway is prevented. Figure 5 (b) shows columns without bracing, which allows sidesway to occur. In practice, the engineer must use best judgement to determine which of the ideal end conditions most closely approximates the actual end conditions to determine proper effective length factors.

Example 1

A column is 11 feet long with one end pinned and the other end fixed. The cross-section is a solid circle with a radius of 2 inches. What is the slenderness ratio? Is the column intermediate or long? What is the allowable stress? The material is steel with *E*=29,000 ksi and yield stress of 36 ksi.

Solution:

The radius of gyration for a solid circular cross-section is defined by *d*/4.

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$$r = \frac{d}{4} = \frac{4\ in}{4} = 1\ in$$

From Figure 4, the effective length factor for a fixed-pinned column is 0.8. Therefore, with an unbraced length of 11 feet the slenderness ratio is
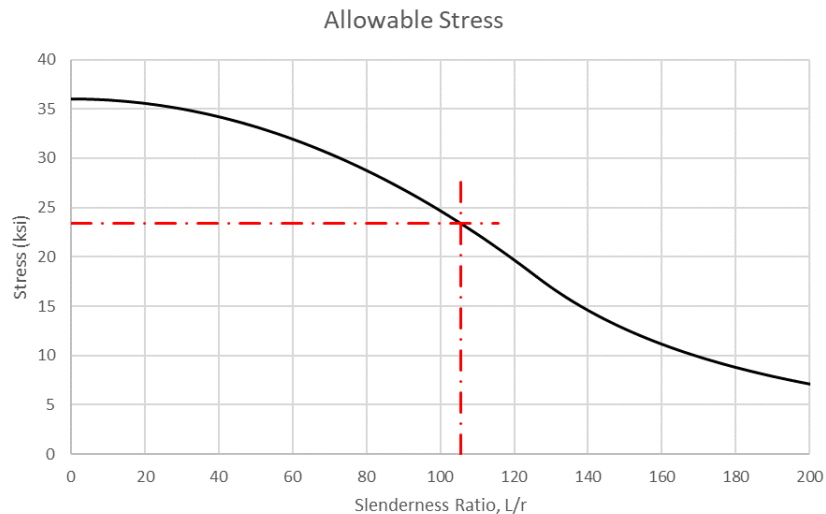
$$\frac{KL}{r} = \frac{0.8(11\ ft)\left(12\ {}^{in}/_{ft}\right)}{1\ in} = 105.6$$

From Equation 6

$$C_c = \sqrt{\frac{2E\pi^2}{\sigma_y}} = \sqrt{\frac{2(29000)\pi^2}{36}} = 126$$

Because the slenderness ratio is less than C_c, the column is intermediate. Using Equation 5

$$\sigma = \left(1 - \frac{(105.6)^2}{2(126)^2}\right)36 = 23.38\ ksi$$

Allowable Stress

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3.0 Basic Column Design

3.1 Introduction

3.1.1 Design Process

Section 2.0 focused on concepts needed for column analysis. We will now move into processes for the design of steel columns. Column design is an iterative trial-and-error type process. It will be seen that allowable stress equations depend on the value of KL/r, where r is a cross-sectional dependent term based on the distribution of the area, but the required area depends on the allowable stress. So, the basic process becomes

1. Choose a column size
2. Determine column stress
3. Compute allowable stress
4. Compare stress from step 2 to allowable stress in step 3
   a. Redesign column if stress is greater than allowable
   b. Redesign if stress is significantly less than allowable to get more efficient design

The process is continued until an appropriate column size is determined. Structural design manuals do have design aids to assist in the process of selecting an initial column size. It is also useful to use computer programs to assist in the iterative process.

3.1.2 Design Codes

This course will present column design procedures in both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) methods. A complete discussion or comparison of these methods is beyond the scope of this course, but a brief description will be presented here. A general comparison of results from both methods will be presented in Section 3.4.

Design of any structural element depends on an accurate estimation of the applied loads. The loading can change, so it is also important to determine the worst possible combination of loadings. For this course, loading will be specified in a combination of dead loads and live loads. Dead loads will include loads that are permanently attached, and they have a constant magnitude and position. Live loads may change position and magnitude.

In ASD, service loads are applied to the structure. In other words, the loads are not factored. ASD formulas for allowable stress are based on reducing the stress by safety factors. In LRFD, the theoretical capacity of the structural member is reduced by a dimensionless resistance factor ϕ, and factored loads are applied to the structure. Multiple load cases are presented in the design manual, but only two will be used in this course. The factors and load cases used here for dead loads (D) and live loads (L) are given below.

$$1.4\,D$$
$$1.2\,D + 1.6\,L$$

Equation 7

The resistance factors used in LRFD design take into account uncertainties in the material's strength and dimensions. The magnitude of the resistance factor is based on loading situations, but the value is 0.85 for columns.

3.2 Column Design with ASD

The design equation used will depend on the slenderness ratio. The critical slenderness ratio C_c calculated from Equation 6 (page 7) will define the transition from long column to intermediate column. For columns with a slenderness ratio less than C_c, the allowable stress is calculated using Equation 8.

$$F_a = \frac{\left(1 - \frac{\left(KL/r\right)^2}{2C_c^2}\right)\sigma_y}{\frac{5}{3} + \frac{3\left(KL/r\right)}{8C_c} - \frac{\left(KL/r\right)^3}{8C_c^3}}$$

Equation 8

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Notice the numerator is simply the previously defined allowable stress from Equation 5 provided on page 7. For columns with a slenderness ratio greater than C_c, the allowable stress is calculated using Equation 9.

$$F_a = \frac{12\pi^2 E}{23\left(KL/r\right)^2} \qquad \text{Equation 9}$$

Note that Equation 9 is the same as Equation 3, which is Euler's formula for buckling, with a safety factor of 23/12 or 1.92.

Figure 6 shows a comparison of the allowable stress determined by the appropriate combination of Equation 8 and Equation 9 using a yield stress of 36 ksi (red line) to the values previously plotted in Figure 2 (black line).
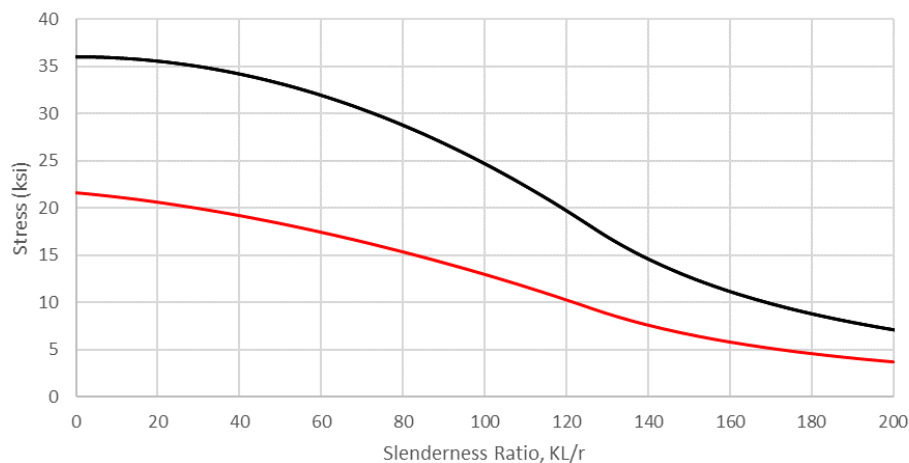

Figure 6   Allowable stress comparison ($E = 29{,}000\ ksi$, $\sigma_y = 36\ ksi$)

Example 2

Select a W12 section for a column used to support a load of 250 kips. The unbraced length is 10 feet and the end conditions are pinned-pinned. Assume a yield stress of 36 ksi.

Solution:

The critical slenderness ratio is

$$C_c = \sqrt{\frac{2E\pi^2}{\sigma_y}} = \sqrt{\frac{2(29000ksi)\pi^2}{36ksi}} = 126$$

Because a member size has not been determined, we don't know the actual slenderness ration (radius of gyration is not yet known). We will begin with an initial guess of a slenderness ratio of $KL/r = 50$. The allowable stress is then determined using Equation 8 because the slenderness ratio is less than 126.

$$F_a = \frac{\left(1 - \frac{(KL/r)^2}{2C_c^2}\right)\sigma_y}{\frac{5}{3} + \frac{3(KL/r)}{8C_c} - \frac{(KL/r)^3}{8C_c^3}}$$

$$F_a = \frac{\left(1 - \frac{(50)^2}{2(126)^2}\right)36ksi}{\frac{5}{3} + \frac{3(50)}{8(126)} - \frac{(50)^3}{8(126)^3}} = 18.35 \ ksi$$

Note that the value of allowable stress can also be determined using Table C-50 from the ASD manual. The required area would be

$$A_{req} = \frac{250 \ kips}{18.5 \ ksi} = 13.62in^2$$

Using tables of W12 properties, the smallest W12 with that area would be a W12x50 (A=14.7, r=1.96). Remember, the allowable stress value was based on an assumption of a slenderness ratio of 50. We now have an actual radius of gyration and must calculate our actual slenderness ratio.

$$\frac{KL}{r} = \frac{1(120in)}{1.96in} = 61.2$$

Recalculating allowable stress using Equation 8 gives

$$F_a = \frac{\left(1 - \frac{(61.2)^2}{2(126)^2}\right)36ksi}{\frac{5}{3} + \frac{3(61.2)}{8(126)} - \frac{(61.2)^3}{8(126)^3}} = 17.31 \ ksi$$

The allowable load is then

$$P = AF_a = 14.7in^2\left(17.31\frac{k}{in^2}\right) = 254.5 \ kips$$

This is greater than the applied load of 250 kips, so a W12x50 is sufficient.

3.3 Column Design with LRFD

LRFD design code proved an equation for the ultimate load as calculated in Equation 10.

$$P_u = \phi_c F_{cr} A_g \qquad\qquad \text{Equation 10}$$

The critical stress term is F_{cr} and the resistance factor $\phi_c = 0.85$ for columns. The factored critical stress is multiplied by gross area to determine ultimate load.

The LRFD design manual provides tables with values of $\phi_c F_{cr}$ for different values of slenderness ratio. Figure 7 shows a comparison of the tabulated $\phi_c F_{cr}$ for a yield stress of 36 ksi and for a yield stress of 50 ksi.

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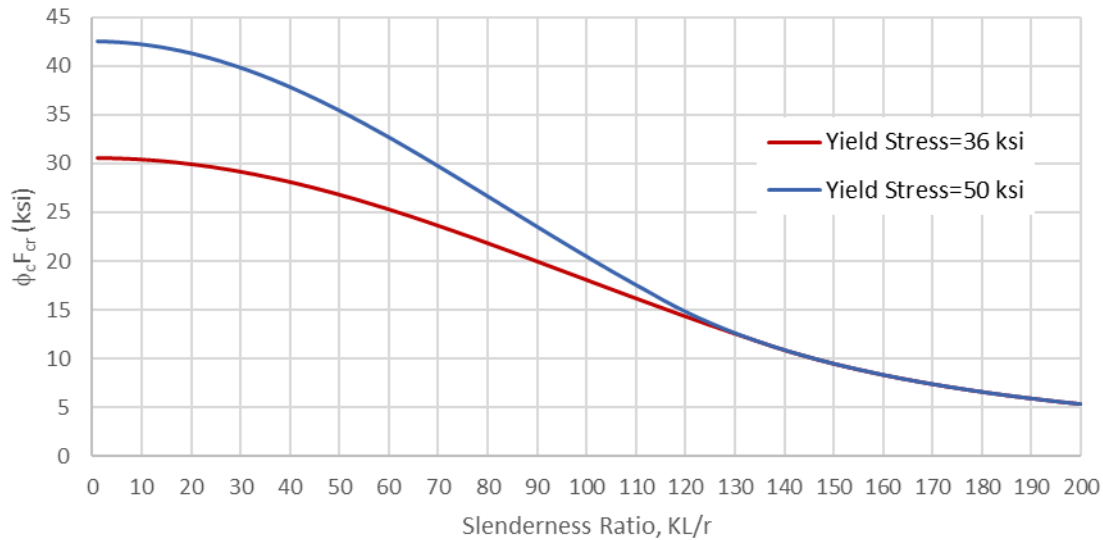*A SunCam online continuing education course*

Figure 7   LRFD allowable stress for $\sigma_y = 36$ ksi and $\sigma_y = 50$ ksi for different slenderness ratios

As stated, values shown in Figure 7 are tabulated in the LRFD design manual, and Section 7.0 of this course provides values for both 36 ksi and 50 ksi steel up to a slenderness ratio of 160. The plot could also be used to approximate allowable stress for specific slenderness ratios.

The following example provides the LRFD procedure for column design. The example also introduces the idea of having different effective lengths for the weak and strong axis of the column. For the problem, the strong axis us unbraced for the entire column length. The weak axis is braced at two intermediate points along the length. For such a problem, the designer must determine the critical effective length and *KL/r* value.

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Example 3

A W14 x 61 column is used for the column shown. The column is braced perpendicular to the weak axis (*y* axis) at the locations shown. The connections are assumed to permit rotation but prevent sidesway. Determine the design strength for the column. Use 36 ksi yield strength.
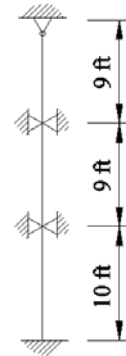
Solution:

Properties for the W14 x 61 are

$$A = 17.9in^2$$

$$r_x = 5.98 \ in$$

$$r_y = 2.45 \ in$$

The effective length for the *x*-axis direction will be based on the entire 28 feet with a fixed support at the bottom and a pinned support at the top. From Figure 4 (b), the effective length factor is 0.8.

$$K_x L_x = 0.8(28 \ ft) = 22.4 \ ft$$

There will be two effective lengths to consider for the *y*-axis. Both 9 ft lengths are pinned-pinned (*K*=1), and the bottom 10 ft length will be fixed-pinned (*K*=0.8).

$$K_y L_y = 1.0(9 \ ft) = 9 \ ft$$

$$K_y L_y = 0.8(10 \ ft) = 8 \ ft$$

Slenderness ratio for the *x*-axis will be

$$\left(\frac{KL}{r}\right)_x = \frac{12(22.4)}{5.98} = 44.95$$

and the worst-case slenderness ratio for the *y*-axis will be

$$\left(\frac{KL}{r}\right)_y = \frac{12(9)}{2.45} = 44.08$$

Therefore, the slenderness ratio in the *x*-axis direction controls the design. Using tables in Section 7.0 along with linear interpolation gives

$$\phi_c F_{cr} = 27.52 \; ksi$$

The design strength is

$$P_u = \phi_c F_{cr} A_g = 27.52 \; ksi(17.9 in^2)$$

$$P_u = 492.6 \; kips$$

3.4 Brief Comparison of ASD to LRFD

A good way to compare the methods for column design would be to compare the stress at service loads. Because LRFD factors the loads, those factors can be removed to compare stresses at service loads. Let's assume that live load equals dead load (LL/DL = 1). Because live load is factored by 1.2 and dead load is factored by 1.6, the total factored load would be divided by 1.4 to get an unfactored service load. Figure 8 shows the allowable stress for ASD compared to LRFD stress divided by the factor of 1.4, plotted as LRFD LF=1.4. If the loading is more dead load compared to live load, the overall load factor would decrease. Such a plot is shown in Figure 8 plotted as LRFD LF=1.3. Load combinations dominated by live load would have higher overall load factors, such as that shown as LRFD LF=1.5.

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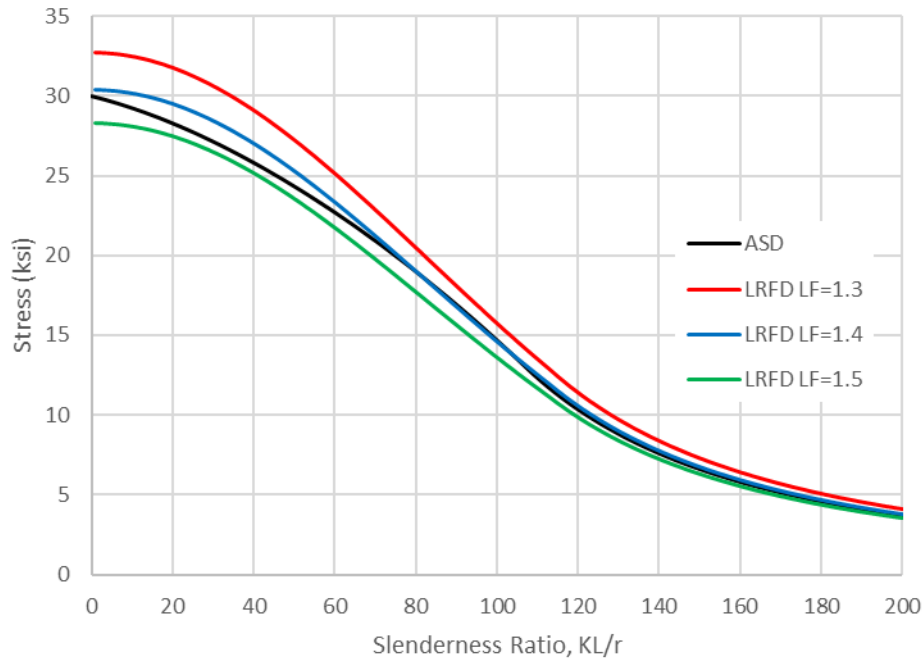
Figure 8   Allowable stress at service loads

It can be seen that if the loading is equally balanced between live load and dead load, the two methods yield very similar results for allowable stress. The LRFD method gives higher allowable stress for conditions with higher dead loads and lower allowable stress as the loading is dominated more by live loads. This is due to the fact that dead loads are easier to predict compared to live loads.

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4.0 Local Buckling

4.1 Introduction

Concepts discussed in Section 3.0 focused on overall stability of the column. However, it is possible for thin flanges or webs to experience local buckling before critical stresses are reached in the entire column. Local buckling conditions for columns are based on equations for plate buckling. A complete coverage of stability of flat plates is beyond the scope of this course, yet a brief coverage is presented here to better understand applications to column local buckling.

4.2 Plate Buckling

To understand local buckling, we must examine plate buckling. Like the Euler formula for critical buckling stress of columns, we can determine critical buckling stress for plates. The critical stress for plates is given by

$$F_{cr} = \frac{k\pi^2 E}{12(1 - \mu^2)\left(b/t\right)^2}$$
Equation 11

where k is a constant based on edge constraints as shown in Figure 9, E is modulus of elasticity, μ is Poisson's ratio, b is the plate's loaded edge length, and t is the plate thickness. Similar to determining slenderness ratios for Euler buckling, Equation 11 can be plotted against different values of b/t to determine limiting values for yielding. Figure 10 shows Equation 11 plotted for K=4, E=30,000 ksi, and μ=0.3. The plot reaches a value of 36 ksi at b/t equal to approximately 55, and it reaches a value of 50 ksi for a b/t equal to approximately 46.6.

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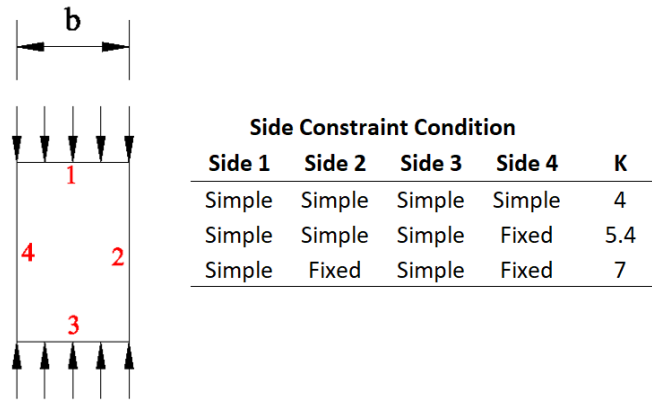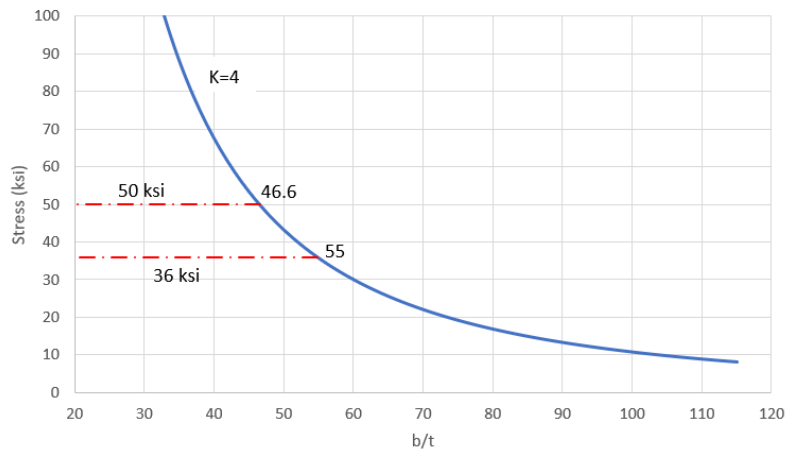*Figure 9*   Plate buckling coefficient, *K*

Figure 10   Critical buckling stress

4.3 Plate Aspect Ratios

Referring back to Figure 9, *b* was defined as the length of the plate along the loaded edge. We can define *a* to be the length of the edge parallel to the loading. The ratio *a/b* is defined as the aspect ratio. As the aspect ratio increases there will be higher modes of buckling, which also

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affects the plate buckling coefficient. Except for cases of extremely short plates, the values of *K* can be used for various aspect ratios with minimal error.

4.4   Compact and Noncompact Sections

Compact sections are those that have sufficient stiffness needed to develop a fully plastic stress (stressed throughout to yield stress) distribution before buckling. Noncompact sections will have a cross-section that allow some areas to reach yield stress, but not all, before buckling occurs. Almost all W sections in the steel manual are compact for both 36 ksi and 50 ksi yield levels.

4.5   Column Applications

Figure 11 shows applications of limiting width to thickness ratios used for wide flange columns. Figure 11 (a) shows the column flange, which is an unstiffened element, and Figure 11 (b) shows the column web, which is a stiffened element. Structural design manuals provide limiting *b/t* ratios for different conditions, such as those shown in Figure 11. The limits are expressed as a value divided by the square root of yield stress.
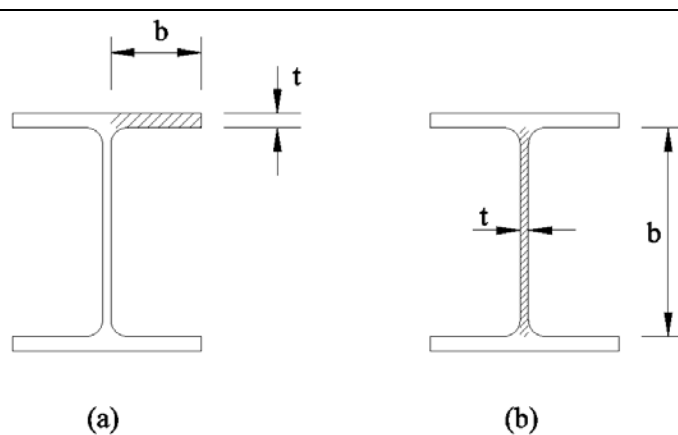

Figure 11   Applications for rolled column flange

5.0   Base Plate and Anchor Bolt Design

5.1   Introduction

5.1.1   Terminology

Base plates are used at the end of the column to distribute the column's compressive load over a larger area. They are the interface between the structure and the foundation. Figure 12 illustrates the components of a typical column base plate. The base plate is typically shop welded to the bottom of the structural column, and they are commonly constructed from ASTM A36 material when possible due to its availability. The type and amount of weld required will depend on the loading, but fillet welds are most common unless the base must resist a large moment (groove welds may then be required). Stiffeners can be included when necessary, but often it is more economical to thicken the base plate rather than adding the complication of stiffeners.
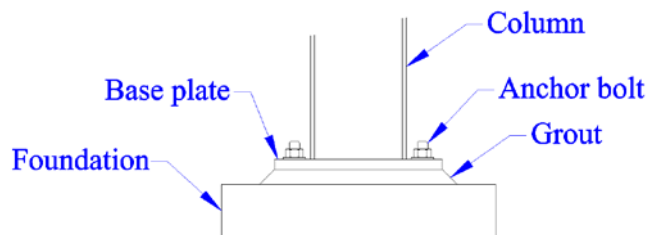
Figure 12   Terminology

Anchor bolts are commonly ASTM F1554. The three grades are 36, 55, and 105 which designate the minimum yield strength (ksi) of the anchor bolt. Grade 36 is the most common, and grade 55 can be used when moment or uplift causes large tension forces in the bolts. The column base plate will have holes for the anchor bolts, and the hole size should be as large as possible to accommodate setting the column over the anchor bolts. Recommended hole sizes, as defined in AISC Manual of Steel Construction (14th edition), are given in Table 1 for different anchor bolt diameters.

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Table 1   Recommended base plate hole and washer sizes

Anchor Dia. (in)	Hole Dia. (in)	Min. Washer Dia. (in)	Min. Washer Thickness (in)
¾	1 5/16	2	1/4
7/8	1 9/16	2 1/2	5/16
1	1 13/16	3	3/8
1 ¼	2 1/16	3	1/2
1 ½	2 5/16	3 1/2	1
1 ¾	2 3/4	4	5/8
2	3 1/4	5	3/4
2 ½	3 1/4	5 1/2	7/8

Table 1 also provides minimum washer diameters, which are based on the ability to cover the entire hole when the anchor is located at an edge of the hole. Washers do not need to be hardened and should not be welded to the base plate except for cases when the anchors must resist shear at the base plate.

Grout, as illustrated in Figure 12, transfers compression loads between the base plate and the foundation. Generally, a 1-inch grout allowance is acceptable if the column will be set on a finished floor. Larger allowance may be used if the column is to be set on a concrete footing.

5.1.2   Loading

Column base plates can have different load cases. The column can have pure axial load, either in compression or tension. A pure concentric compression load is the most basic, and the tension loading would cause uplift at the base plate. The column base plate could also be loaded in shear or have moment loadings. Section 5.2 will discuss load cases with concentric compression loading and Section 5.3 will focus on the general idea of designing base plates for moment loads. Uplift and shear loadings are beyond the scope of this course.

5.2 Compressive Axial Load

The first loading case considered will be that of pure axial compression. For this basic loading case, the base plate must have a large enough area to not exceed the concrete bearing limit. The base plate must also have adequate thickness to prevent base plate yielding.

For a case when the base plate covers the entire area of concrete (no concrete confinement), the nominal bearing stress is

$$P_p = 0.85 f_c' A_1 \qquad \text{Equation 12}$$

and if the concrete area is not covered (concrete confinement)

$$P_p = 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}}$$

$$\text{Where } \sqrt{\frac{A_2}{A_1}} \leq 2$$

Equation 13

In both equations, A_1 is the area of the base plate and A_2 is the maximum area of the supporting surface having the same aspect ratio as the base plate. For LRFD, Equation 12 and Equation 13 would be multiplied by a resistance factor $\phi_c = 0.65$ to get $P_u = \phi_c P_p$, or they would be divided by a safety factor $\Omega = 2.5$ to get $P_a = \frac{P_p}{\Omega}$, when using ASD.

Essentially, the two equations above are identical. For a case when there is no concrete confinement, A_2 will equal A_1 so the square root term would just reduce to unity. In general, there will be three possible design cases:

Case I $\qquad A_2 = A_1$

Case II $\qquad A_2 \geq 4A_1$

Case III $\qquad A_1 < A_2 < 4A_1$

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For this fundamentals course, we will only discuss Case I, which is when there will be no concrete confinement. This case is the most conservative. The required base plate area A_1 would be determined by rearranging Equation 12 and using the appropriate safety factor or resistance factor.

$$LRFD \qquad (A_1)_{req} = \frac{P_u}{\phi_c 0.85 f_c'} \qquad \text{Equation 14}$$

$$ASD \qquad (A_1)_{req} = \frac{\Omega P_a}{0.85 f_c'} \qquad \text{Equation 15}$$

With the required base plate area determined, the base plate dimensions can be calculated. Figure 13 illustrates the dimensions. The column depth is defined by *d* and the flange width is defined by b_f.
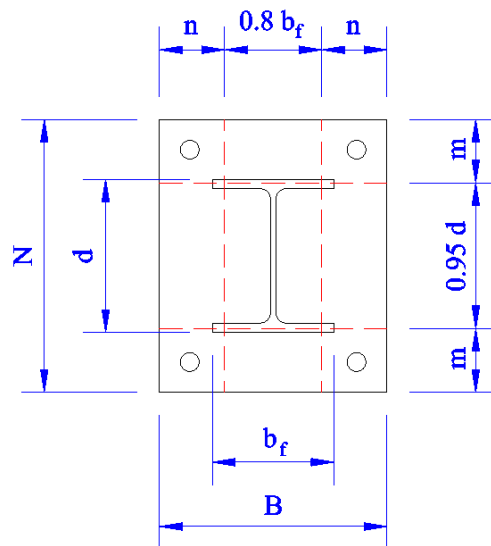
Figure 13   Base plate dimensions

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The dimensions of the base plate should be such that *m* and *n*, as shown in Figure 13, are approximately equal. To optimize the base plate dimensions

$$\Delta = \frac{0.95d - 0.8b_f}{2}$$

Equation 16

and the optimal base plate length is

$$N \sim \sqrt{(A_1)_{req}} + \Delta$$

Equation 17

The value used for *N* can then be rounded to a desired value. The plate width is then determined from

$$B = \frac{(A_1)_{req}}{N}$$

Equation 18

Next, we can calculate the base plate thickness. The overhanging dimensions shown in Figure 13 are defined as

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

Equation 19

$$n = \frac{B - 0.8b_f}{2}$$

Equation 20

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One additional critical dimension is considered based on plate yield line theory. As a plate is loaded, the initial response will be purely elastic behavior. At some level of loading, local yielding will occur. As the points of local yielding continue, they will form a line called a yield line. Thornton (Thornton, 1990) proposed that the plate thickness be determined using the maximum value of *m* (from Equation 19), *n* (from Equation 20), or $\lambda_{n'}$, which is defined by

$$\lambda_{n'} = \lambda \frac{\sqrt{db_f}}{4} \qquad \text{Equation 21}$$

where

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1 \qquad \text{Equation 22}$$

and

$$LRFD \qquad X = \left(\frac{4db_f}{(d + b_f)^2}\right) \frac{P_u}{\phi_c P_p} \qquad \text{Equation 23}$$

$$ASD \qquad X = \left(\frac{4db_f}{(d + b_f)^2}\right) \frac{\Omega P_a}{P_p} \qquad \text{Equation 24}$$

The plate's cantilever distance used for thickness calculations will be the maximum of all cases.

$$l = \max(m, n, \lambda_{n'}) \qquad \text{Equation 25}$$

The minimum plate thickness is then calculated.

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$$LRFD \qquad t_{min} = l\sqrt{\dfrac{2P_u}{\phi F_y BN}} \qquad \text{Equation 26}$$

$$ASD \qquad t_{min} = l\sqrt{\dfrac{2\Omega P_a}{F_y BN}} \qquad \text{Equation 27}$$

Example 4

Consider a W10 x 45 column support a live load of 120 kips and a dead load of 60 kips. The concrete compressive strength is 3 ksi. Using the assumption that $A_2 = A_1$, determine the required base plate dimensions using the LRFD method and a plate yield strength of 36 ksi.

Solution:

The factored load is determined using Equation 7 to get

$$P_u = 1.2(60\ k) + 1.6(120\ k)$$

$$P_u = 264\ k$$

The required area would be

$$(A_1)_{req} = \frac{P_u}{\phi_c 0.85 f_c'}$$

$$(A_1)_{req} = \frac{264k}{0.65(0.85)(3\ ksi)} = 159.28\ in^2$$

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Next, we can optimize the baseplate dimensions. A W10 x 45 has a depth of 10.125 inches and a flange width of 8 inches.

$$\Delta = \frac{0.95d - 0.8b_f}{2}$$

$$\Delta = \frac{0.95(10.125\ in) - 0.8(8\ in)}{2} = 1.61\ in$$

$$N \sim \sqrt{(A_1)_{req}} + \Delta$$

$$N \sim \sqrt{159.28 in^2} + 1.61in \sim 14.23\ in \ \ \text{use 15 inches}$$

$$B = \frac{(A_1)_{req}}{N}$$

$$B = \frac{159.28\ in^2}{15\ in} = 10.6\ in \ \ \text{use 11 inches}$$

Check capacity

$$\phi_c P_p = \phi_c 0.85 f_c' A_1 = 0.65(0.85)(3ksi)(15in)(11in)$$

$$\phi_c P_p = 273.5\ kips > 264\ kips$$

Next, we determine required plate thickness.

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

$$m = \frac{15 - 0.95(10.125)}{2} = 2.69\ in$$

$$n = \frac{B - 0.8b_f}{2}$$

$$n = \frac{11 - 0.8(8)}{2} = 2.3\ in$$

$$X = \left(\frac{4db_f}{(d + b_f)^2}\right)\frac{P_u}{\phi_c P_p}$$

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$$X = \left(\frac{4(10.125)(8)}{(10.125 + 8)^2}\right)\frac{264\ kips}{273.5\ kips} = 0.95$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$$

$$\lambda = \frac{2\sqrt{0.95}}{1 + \sqrt{1 - 0.95}} = 1.6$$

Therefore, use $\lambda = 1$.

$$\lambda_{n'} = \lambda\frac{\sqrt{db_f}}{4} = 1\frac{\sqrt{10.125(8)}}{4} = 2.25\ in$$

$$l = \max(m, n, \lambda_{n'}) = 2.69\ in$$

The minimum thickness is calculated next, using the resistance factor for base plate bending of 0.9.

$$t_{min} = l\sqrt{\frac{2P_u}{\phi F_y BN}}\ l = \sqrt{\frac{2(264\ kips)}{0.9(36\ ksi)(11\ in)(15\ in)}} = 0.85\ in$$

Continuation of the example problem would include rounding the plate thickness up to the next practical value. No anchor bolt forces exist for this application, so anchor bolt diameters can be determined using minimum values defined by code and practicality. When using wide flange columns subjected only to compressive axial loading, the base plate is commonly welded to the column with fillet welds on one side of each flange only. For rectangular HSS columns, a fillet weld on each flat side is common.

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5.3 General Concepts of Moment Loadings

5.3.1 Introduction

This section will briefly discuss concepts associated with base plate design for moment loads. Full coverage of the topic is beyond the scope of this course as it requires knowledge of welded connection design and anchor bolt pullout calculations.

5.3.2 Load Eccentricity

Axial loads will cause compressive stress between the base plate and the footing, as discussed in Section 5.2. A moment load will increase the compressive stress on one side of the base plate while decreasing it on the other.

Consider the loading illustrated in Figure 14 (a). The column is loaded with a compressive load P at the centerline of the column and a moment load M. The column is loaded in axial compression and bending, so the forces transferred to the base plate will be a result of that combined loading.
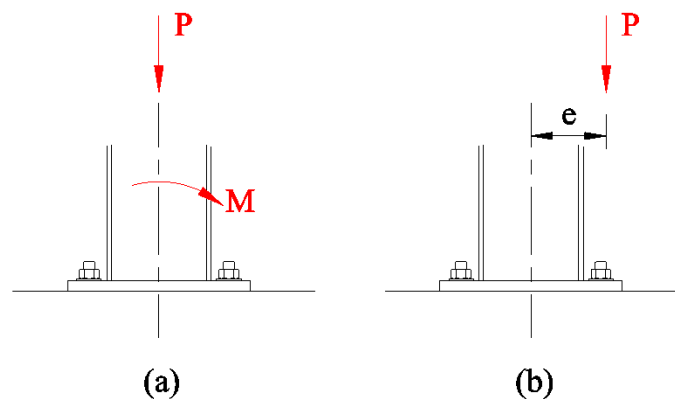
Figure 14   (a) Column loaded with a moment (b) Equivalent eccentric loading

![SunCam logo]

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The common procedure is to replace the loads with an equivalent eccentric loading as shown in Figure 14 (b). The axial load is the same magnitude as that shown in Figure 14 (a). The eccentricity distance needed to develop the equivalent moment load is

$$e = \frac{M}{P}$$
 Equation 28

The design procedure used for the base plate will depend on the magnitude of the eccentricity *e* compared to the length dimension of the base plate *N* (as defined in Figure 13). Small eccentricity, as shown in Figure 15 (a), will maintain compressive stress between the base plate and the foundation. Larger eccentricity, as shown in Figure 15 (b), will result in uplift that must be resisted by anchor bolt tension.

Figure 15   (a) Small eccentricity (b) Large eccentricity

Appropriate design procedure will be based on a critical value of eccentricity. The magnitude of the critical eccentricity is the base plate length *N* divided by 6.
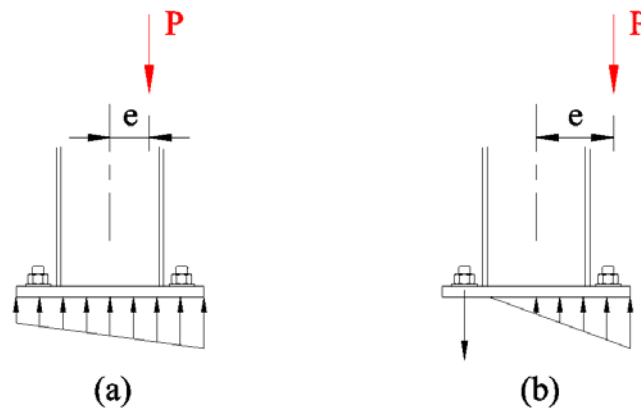
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$$e_{cr} = \frac{N}{6}$$
<div align="right">Equation 29</div>

If the actual eccentricity is less than the critical value, use the procedures for small moments. If the actual eccentricity is greater than the critical value, the procedures for large moments will be used.

The general design procedure is as follows:

1. Determine the axial load and moment for the base plate
2. Determine equivalent eccentricity (Equation 28)
3. Pick a trial base plate size N x B
4. Calculate critical eccentricity (Equation 29)
5. Calculate required base plate thickness
    a. If $e \leq e_{cr}$ follow procedure for small moments (Section 5.3.3)
    b. If $N/2 > e > e_{cr}$ follow procedure for large moments (Section 5.3.4)
6. Determine required anchor bolt size
7. Design weld between column and base plate

The design process above is often iterative and may require adjustments to base plate dimensions. During the design it must be verified that concrete bearing pressure does not exceed maximum allowed values. When uplift exists, the anchor bolt design must include calculations of pullout strength and embedment.

5.3.3   Small Moments

When the eccentricity, calculated from Equation 28, is less than the critical value defined in Equation 29, the base plate force distribution is similar to that shown in Figure 15 (a). The bearing stress distribution will be a superposition of axial stress and bending stress.

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I}$$
<div align="right">Equation 30</div>

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Defining the base plate width as *B* and the length as *N*, Equation 30 becomes

$$\sigma = \frac{P}{BN} \pm \frac{M(N/2)}{1/12\,BN^3}$$

$$\sigma = \frac{P}{BN} \pm \frac{6M}{BN^2} \qquad \text{Equation 31}$$

As a quick justification of Equation 29, when the eccentricity *M/P* equals the critical value of *N*/6, Equation 31 becomes $\sigma = \frac{P}{BN} \pm \frac{P}{BN}$ giving a pressure of zero on one end. The bearing stress from Equation 31 gives a general condition as shown in Figure 16 (a).

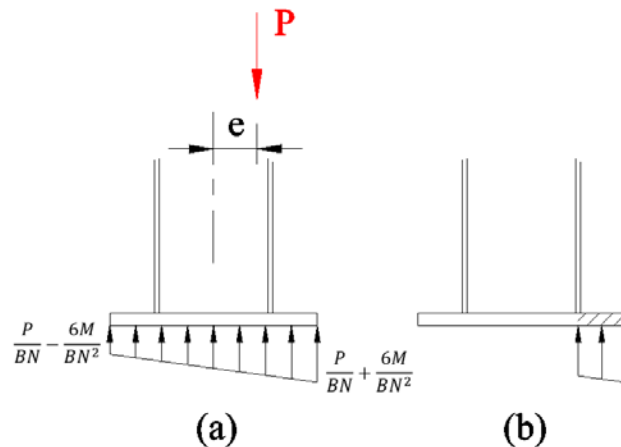*Figure 16*   Stress distribution for small moments

A common procedure for calculating required base plate thickness is to use the stress distribution shown in Figure 16 (b) as loading on the shaded cantilever portion of the base plate. The bending moment is calculated in the base plate for a one-inch strip cantilevered beam. The thickness is determined by equating the bending moment to the plate's yield stress.

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$$\sigma_y = \frac{Mc}{I}$$

$$M = \frac{\sigma_y I}{c} = \frac{\sigma_y \, {}^1/_{12}\,(1\ in)t^3}{{}^t/_2}$$

Rearranging the equation gives the equation for minimum base plate thickness.

$$t_{min} = \sqrt{\frac{6M}{\sigma_y}} \qquad\qquad \text{Equation 32}$$

The moment in Equation 32 is determined from the bearing stress loading shown in Figure 16 (b) at the location in the base plate at the center of the right flange. Resistance factors or safety factors need to be applied based on LRFD or ASD specifications.

5.3.4   Large Moments

Once the eccentricity exceeds the critical value, the base plate force distribution is similar to that shown in Figure 17 (a). For this loading, the anchor bolts on one side will have tensile load T due to uplift. The bearing stress will decrease linearly to a distance Y from the edge of the base plate.

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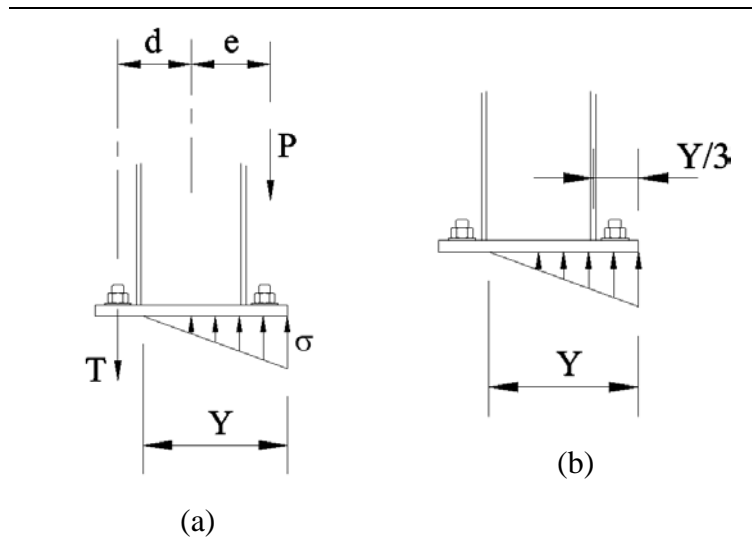
Figure 17   Large moment forces

Using a bearing plate width of *B*, summation of vertical forces from Figure 17 (a) gives

$$P + T = \frac{\sigma Y B}{2}$$ Equation 33

Equation 33 has three unknowns of *T*, *Y*, and σ. A common simplifying assumption is that the stress distribution's center of gravity (*Y*/3) coincides with the center of the column flange, as illustrated in Figure 17 (b). This assumption reduces the equation to two unknowns, which requires a second equation. The second equation comes from a summation of moments of forces shown in Figure 17 (a) about the column's centerline.

Once all forces are known, the base plate can be treated as a cantilever beam to determine plate thickness. The process is similar to that discussed in Section 5.3.3. The welded joint between the column and base would need to be designed, anchor bolt pullout would need to be analyzed, and it would need to be confirmed that allowable concrete bearing has not been exceeded.

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6.0   References and Further Reading

Thornton, W. A. (1990). Design of Base Plates for Wide Flange Columns - A Concatenation of Methods. *Engineering Journal AISC*, 173-174.

Blodgett, O. W. (1966). Design of Welded Structures. *James F. Lincoln Arc Welding Foundation*

Fisher, J. M. and Kloiber, L. A. (2006). Base Plate and Anchor Rod Design. *AISC*

7.0   LRFD Tables

KL/r	$\phi_c F_{cr}$ 36 ksi	50 ksi	KL/r	$\phi_c F_{cr}$ 36 ksi	50 ksi	KL/r	$\phi_c F_{cr}$ 36 ksi	50 ksi	KL/r	$\phi_c F_{cr}$ 36 ksi	50 ksi
1	30.60	42.50	41	28.01	37.59	81	21.66	26.31	121	14.16	14.57
2	30.59	42.49	42	27.89	37.36	82	21.48	26.00	122	13.98	14.33
3	30.59	42.47	43	27.76	37.13	83	21.29	25.68	123	13.80	14.10
4	30.57	42.45	44	27.64	36.89	84	21.11	25.37	124	13.62	13.88
5	30.56	42.42	45	27.51	36.65	85	20.92	25.06	125	13.44	13.66
6	30.54	42.39	46	27.37	36.41	86	20.73	24.75	126	13.27	13.44
7	30.52	42.35	47	27.24	36.16	87	20.54	24.44	127	13.09	13.23
8	30.50	42.30	48	27.11	35.91	88	20.36	24.13	128	12.92	13.02
9	30.47	42.25	49	26.97	35.66	89	20.17	23.82	129	12.74	12.82
10	30.44	42.19	50	26.83	35.40	90	19.98	23.51	130	12.57	12.62
11	30.41	42.13	51	26.68	35.14	91	19.79	23.20	131	12.40	12.43
12	30.37	42.05	52	26.54	34.88	92	19.60	22.89	132	12.23	12.25
13	30.33	41.98	53	26.39	34.61	93	19.41	22.58	133	12.06	12.06
14	30.29	41.90	54	26.25	34.34	94	19.22	22.28	134	11.88	11.88
15	30.24	41.81	55	26.10	34.07	95	19.03	21.97	135	11.71	11.71
16	30.19	41.71	56	25.94	33.79	96	18.84	21.67	136	11.54	11.54
17	30.14	41.61	57	25.79	33.51	97	18.65	21.36	137	11.37	11.37
18	30.08	41.51	58	25.63	33.23	98	18.46	21.06	138	11.20	11.20
19	30.02	41.39	59	25.48	32.95	99	18.27	20.76	139	11.04	11.04
20	29.96	41.28	60	25.32	32.67	100	18.08	20.46	140	10.89	10.89

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21	29.90	41.15	61	25.16	32.38	101	17.89	20.16	141	10.73	10.73
22	29.83	41.02	62	24.99	32.09	102	17.70	19.86	142	10.58	10.58
23	29.76	40.89	63	24.83	31.80	103	17.51	19.57	143	10.43	10.43
24	29.69	40.75	64	24.67	31.50	104	17.32	19.28	144	10.29	10.29
25	29.61	40.60	65	24.50	31.21	105	17.13	18.98	145	10.15	10.15
26	29.53	40.45	66	24.33	30.91	106	16.94	18.69	146	10.01	10.01
27	29.45	40.29	67	24.16	30.61	107	16.75	18.40	147	9.87	9.87
28	29.36	40.13	68	23.99	30.31	108	16.56	18.12	148	9.74	9.74
29	29.28	39.97	69	23.82	30.01	109	16.37	17.83	149	9.61	9.61
30	29.18	39.79	70	23.64	29.70	110	16.19	17.55	150	9.48	9.48
31	29.09	39.62	71	23.47	29.40	111	16.00	17.27	151	9.36	9.36
32	28.99	39.43	72	23.29	29.09	112	15.81	16.99	152	9.23	9.23
33	28.90	39.25	73	23.12	28.79	113	15.63	16.71	153	9.11	9.11
34	28.79	39.06	74	22.94	28.48	114	15.44	16.42	154	9.00	9.00
35	28.69	38.86	75	22.76	28.17	115	15.26	16.13	155	8.88	8.88
36	28.58	38.66	76	22.58	27.86	116	15.07	15.86	156	8.77	8.77
37	28.47	38.45	77	22.40	27.55	117	14.89	15.59	157	8.66	8.66
38	28.36	38.24	78	22.22	27.24	118	14.70	15.32	158	8.55	8.55
39	28.25	38.03	79	22.03	26.93	119	14.52	15.07	159	8.44	8.44
40	28.13	37.81	80	21.85	26.62	120	14.34	14.82	160	8.33	8.33