CVEN 446
Structural Steel Design

Module 17

Design of Steel Connections II

April 22, 2015

Moment Connections

A moment connection is designed to transfer the full design moment with little or no rotation of the joint, in addition to the design shear force.

The web carries the shear. Since relative rotation between the beam and column is limited, there is no significant eccentricity on the beam web bolt pattern.

A single plate shear tab can be used and would be designed in the same manner as previously discussed with the exception of the bolt eccentricity.

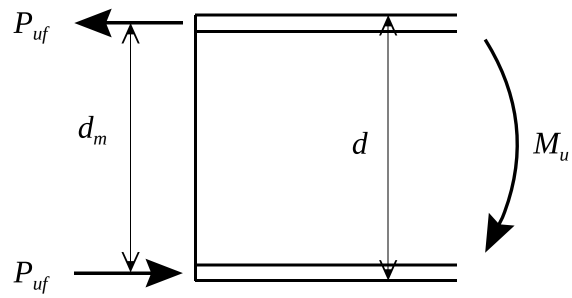
Force - Couple Resolution

The bending moment is carried by the flanges and can be resolved into a force couple.

The compression and tension flange forces are then given by:

$$P_{uf} = \frac{M_u}{d_m}$$

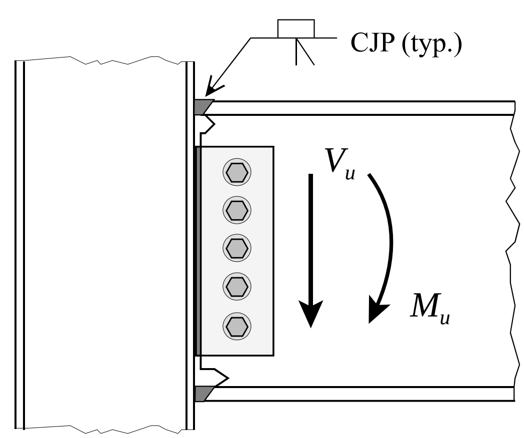
where d_m is the moment arm and differs depending on the type of connection.



Welded Flange Moment Connection

The simplest beam-to-column moment connection is the welded flange plate connection in which the flanges of the beam are butt welded directly to the column.

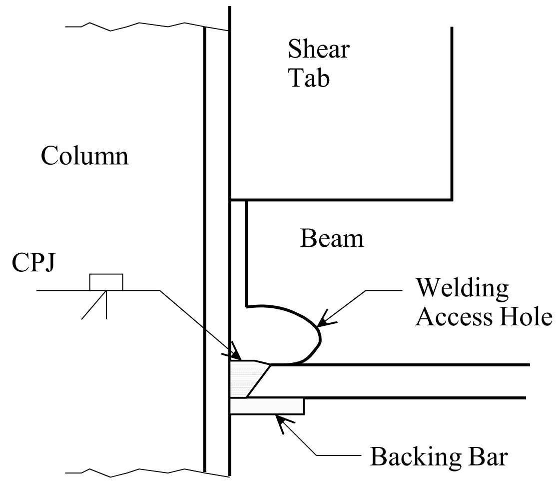
The shear is carried by a single plate shear tab, designed as previously discussed except that the eccentricity on the bolt pattern can be ignored.



Welded Flange Moment Connection

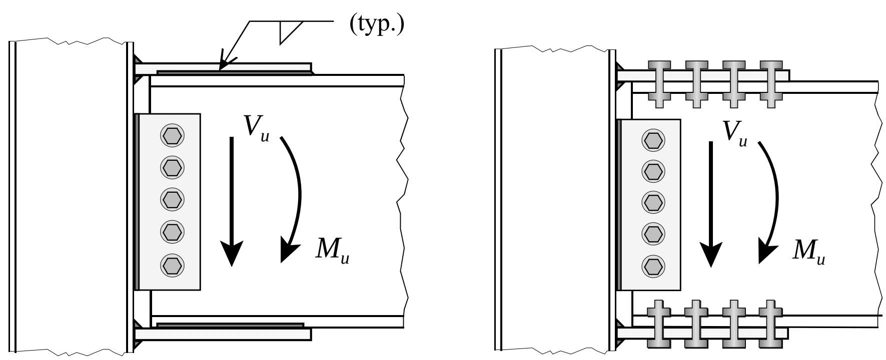
Since the weld is a full penetration groove weld, all that is required for its design is that weld metal matching the beam metal be used. A backing bar is used to contain the weld metal.

Since the moment will reverse with the direction of the horizontal loading, both the top and bottom welds are subject to tension and are both critical.



Flange-Plated Moment Connection

The beam flange forces can also be transferred to the column using either welded or bolted flange plates.



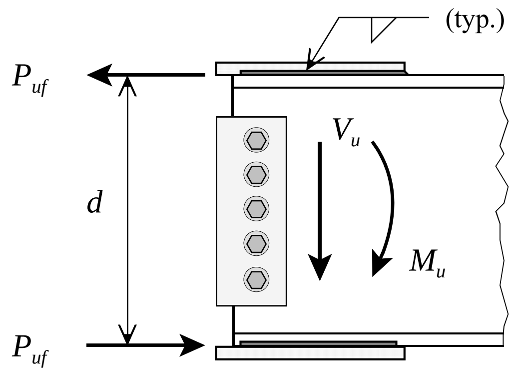
The shear can be carried by a single plate shear tab.

Welded Flange-Plated Moment Connection

The compression and tension flange forces are determined by using the full depth of the beam section:

$$P_{uf} = \frac{M_u}{d}$$

Required weld length per plate is based on P_{uf} and the chosen weld size.

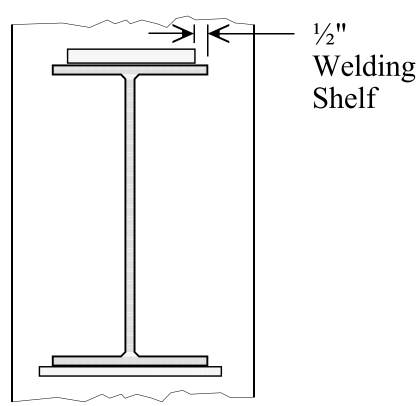


Welded Flange-Plated Moment Connection

Use different plate widths to eliminate overhead welding.

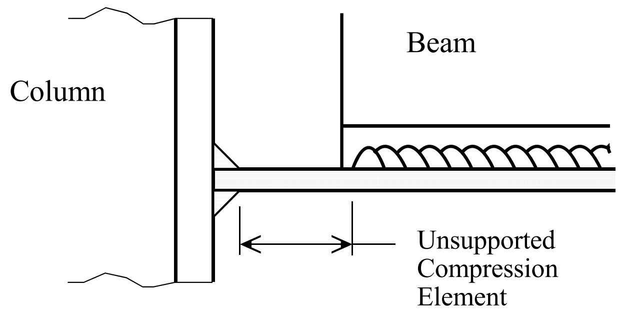
A ½" shelf should be maintained so that there is adequate space for the fillet welds.

If end welds are used, the plates are sized according to only Gross Section Yielding (GSY).



Welded Flange-Plated Moment Connection

Since the bottom plate is usually subject to compression, it should be check for buckling between the end of the longitudinal fillet weld attaching the plate to the beam and the transverse fillet welds attaching the plate to the column.



Welded Flange-Plated Moment Connection - Design

1) Determine the required flange force.

2) Determine the width of the top flange plate:

$$b \quad \leq \quad b_f \text{ - } 2(0.5")$$

3) Determine the width of the top plate based on gross section yielding:

$$t_{req} \geq \frac{P_{uf}}{0.9 F_y b}$$

4) Select an appropriate weld size ($a \leq t_p$ - 1/16") and determine the required weld length (use end weld).

Welded Flange-Plated Moment Connection - Design

5) Determine the width of the bottom flange plate:

$$b \quad \geq \quad b_f + 2(0.5")$$

6) Determine the width of the bottom plate based on gross section yielding:

$$t_{req} \geq \frac{P_{uf}}{0.9 F_y b}$$

5) Select an appropriate weld size ($a \leq t_p$ - 1/16") and determine the required weld length (use end weld).

6) Check Net Section Rupture (NSR) assuming an end weld is not used as it would require an overhead weld.

Welded Flange-Plated Moment Connection - Design

7) Determine the size of the fillet weld attaching the plate to the column flange:

$$a_{req} = \frac{P_{uf}}{0.75(0.6) F_{EXX}\, 0.707(2) b}$$

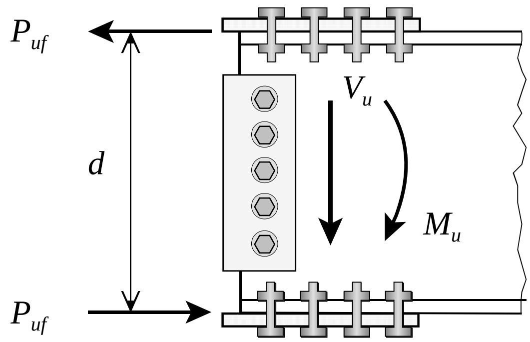
8) Check buckling of the bottom plate. Assume $K = 0.65$ and an unsupported length of 1-1/2".

Bolted Flange-Plated Moment Connection

The compression and tension flange forces are determined by using the full depth of the beam section:

$$P_{uf} = \frac{M_u}{d}$$

Plates of equal width should be used.

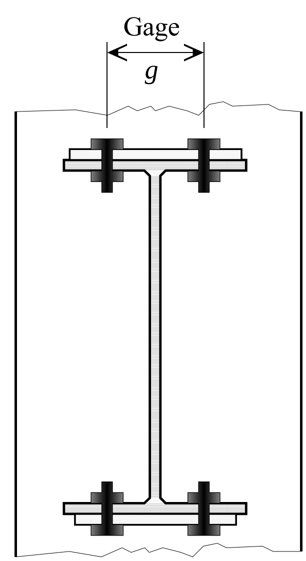


Bolted Flange-Plated Moment Connection

The flange plates are designed for both Gross Section Yielding (GSY) and Net Section Rupture (NSR, $U = 1.0$).

A double row of bolt are used and consideration of the gage distance is necessary for adequate clearance for tightening.

The bolts are subject to single shear conditions.



Bolted Flange-Plated Moment Connection

The short tension components, such as flange and splice plates, the net area must not exceed 85 percent of the gross area, or:

$$A_n \quad \leq \quad 0.85A_g$$

This requirement is to insure ductility as the component length is short and the inelastic deformation limited.

Note that the actual net area can be greater than 85 percent of the gross area. Only that no more than 85 percent can be used in the calculation of the tension capacity based on net section Rupture (NSR).

Bolted Flange-Plated Moment Connection - Design

1) Determine the required flange force.

2) Determine the required number of single shear bolts. Round to the next even whole bolt. Adjust bolt diameter to obtain a reasonable number.

3) Select plate thickness, *b*. Should not be wider than the larger of either the beam or column flange.

4) Determine the plate thickness based on Gross Section Yielding

5) Check Net Section Fracture.

Bolted Flange-Plated Moment Connection - Design

6) Check Block Shear Rupture of the flanges.

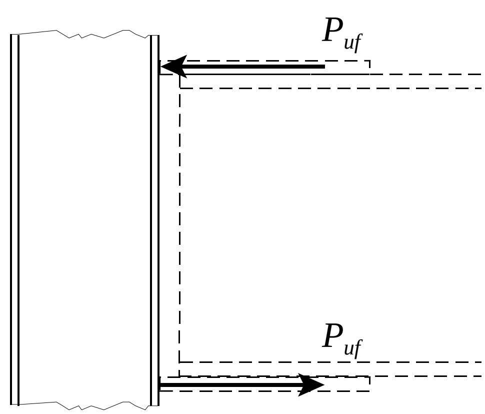
7) Check bolt bearing in both the plate and beam flange.

8) Determine the size of the fillet weld attaching the plate to the column flange:

$$a_{req} = \frac{P_{uf}}{0.75(0.6)F_{EXX}\,0.707(2)b}$$

Column Concentrated Forces

The beam flange connection delivers concentrated forces that must be adequately resisted by the column.



Column Concentrated Forces

Tension Force

- Flange Local Bending
  (see AISC-14 Sec. J10.1)
- Web Local Yielding
  (see AISC-14 Sec. J10.2)

Compression Force

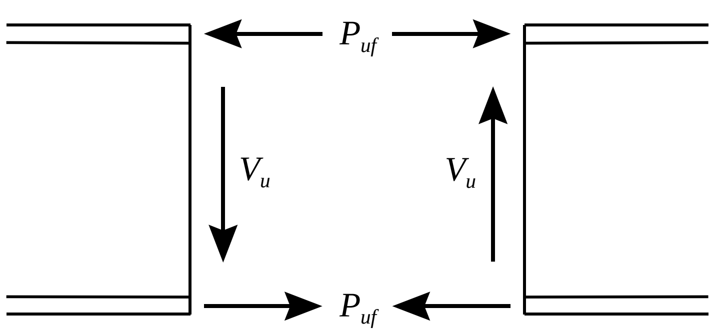
- Web Crippling
  (see AISC-14 Sec. J10.3)
- Web Compression Buckling
  (see AISC-14 Sec. J10.5)

Column web stiffeners or doubler plates may be required.

Beam Moment Splices

A beam splice may be required when the span length becomes long or is required for erection purposes.

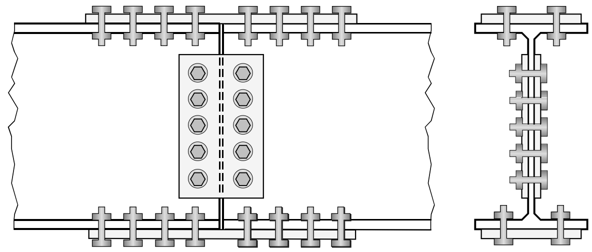
The beam splice must transfer the shear force and bending moment between the ends of the beams.



Beam Moment Splices

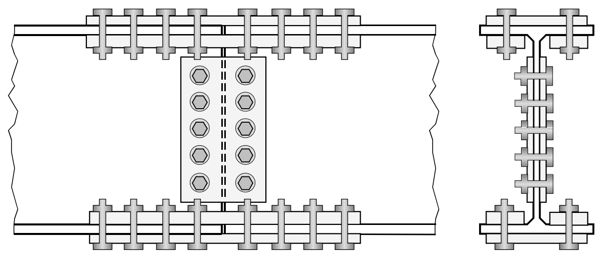
4 - Plate Splice

Flange bolts subject to single shear



8 - Plate Splice

Flange bolts subject to double shear



Beam Moment Splices

The bolts transferring the shear in the web need not be designed for eccentric loading as the flange plates prevent relative rotation.

The splices need only develop the strength required by the forces at the point of the splice.

When a relatively large moment must be transferred, an eight-plate splice becomes preferable as the double shear bolts reduce the overall number of required bolts for the flanges.

A double web splice is always used.

Column Base Plates

Column base plates serve to transfer the load between the steel building frame and the concrete foundation.

Since the compressive strength of steel is significantly higher than the concrete, the axial load in the column must to spread out over a greater area on the concrete.

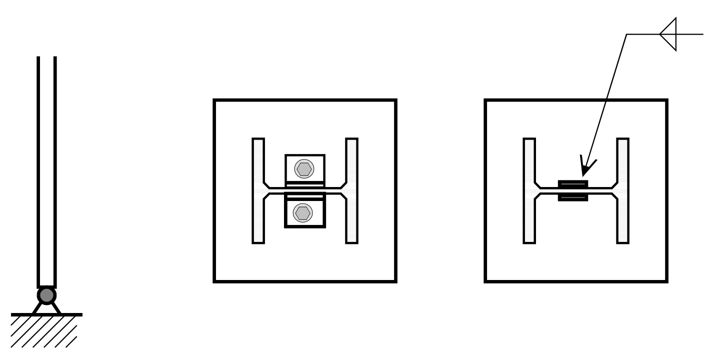
There are two types of column base connections:

1) pinned
2) fixed

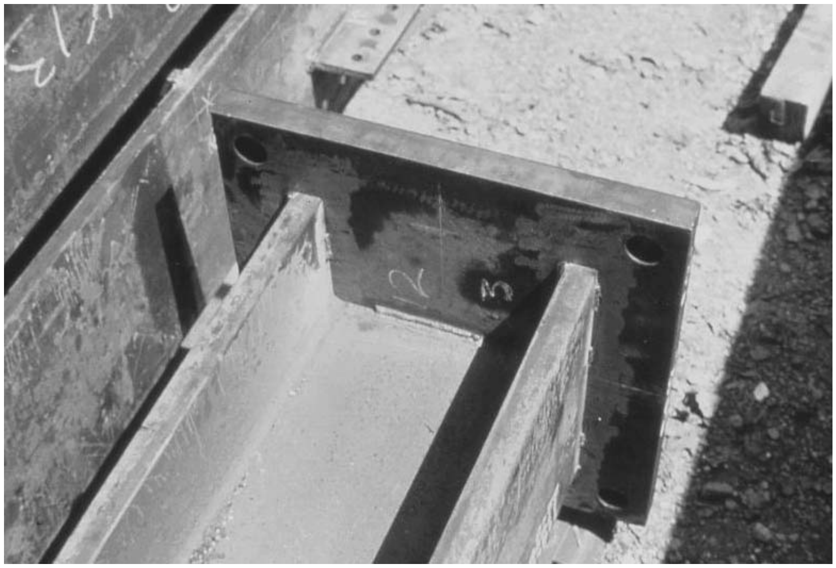
Note that using a fixed column base requires the column footing to be designed for moment in addition to axial load.

Column Base Plates - Pinned

The pinned column base connection transfers only axial loads to the foundation. To insure that an axial load is transferred and not bending moment, a connection between only the web and the base plate is made.



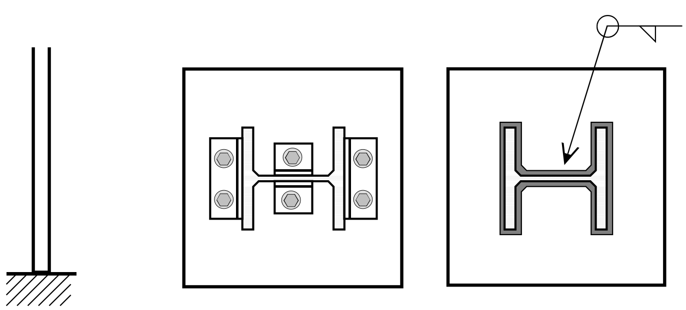
Column Base Plates - Pinned



Column Base Plates - Fixed

The fixed column base connection is designed to transfer both axial loads and bending moment to the foundation.

The column flanges are attached, as well as the web, leading to the development of the flange forces of the bending moment.



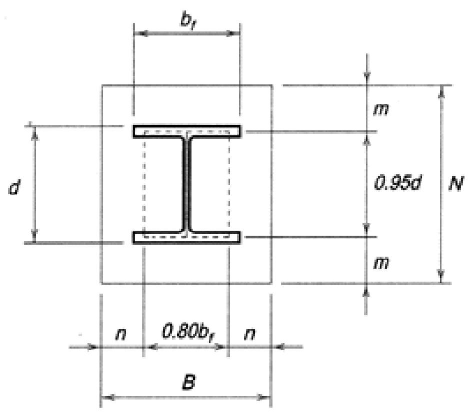
Column Base Plates - Design (AISC-14 Sec. 14)

A column base plate design must consider the following:

1. Plate area to reduce the bearing stress on the supporting concrete to an acceptable level.

2. Sufficient plate thickness to prevent bending failure.

AISC uses the following nomenclature:



Column Base Plates - Design

The area of the base plate is given as:

$$A_1 = N \text{ x } B$$

When the plate is the same size as the supporting concrete:

$$\phi P_p = 0.65(0.85)\, f_c^{'} A_1$$

When the plate is less than the full aea of the concrete support:

$$\phi P_p = 0.65(0.85)\, f_c^{'} A_1 \sqrt{\frac{A_2}{A_1}} \leq 0.65(1.7)\, f_c^{'} A_1$$

where A_2 is the maximum area of the support that is geometrically the same as the plate, up to four times the area of the plate.

Column Base Plates - Design

As the base plate cantilevers out beyond the column section, the plate must be designed for bending. The largest of the following cantilever dimensions is taken as *l*:

$$n = \frac{B - 0.80 b_f}{2} \qquad m = \frac{N - 0.95 d}{2}$$

$$\lambda n' = \lambda \frac{\sqrt{b_f d}}{4}$$

where λ can be conservatively taken as 1.0 (see AISC-14 p. 14-5).

The minimum plate thickness is then given as:

$$t_{\min} = l \sqrt{\frac{2 P_u}{0.9 F_y B N}}$$