

Bolted Beam-Column Connections (Design and Cost Estimation)

Abstract

One of the most favorable features of steel structure is reducing the labor required on site. This is achieved by using bolted connections. Obviously, if lots of field weld is required, the benefit of fast installation will be reduced. Bolted connections are the most commonly used ones in modern steel constructions. If sorted by beam-column profiles, bolt types, connection types and the forces to be transferred, there could be hundreds of bolted beam-column connections while only some of them are commonly used. A properly designed bolted connection is able to transfer shear force and/or moment and tension. To fulfill the force transfer, quite a few items need to be checked. Different connections require different items to be checked. But some common calculations is required for almost all bolted connections, such as bolt shear, bolt tension, connection member bearing, tension and shear block etc.

This paper will list the most commonly used bolted beam-column connections and discuss the characteristics and usual design procedure of these connections. Then an example will be explained more detailed and a spreadsheet will be developed for the design of this connection. It is worth to be noted that although the spreadsheet is developed for the particular connection, it can also be used for other connections to check the same working circumstances or further development could be made accordingly.

1. Introduction

Usually two types of forces are transferred by bolted beam-column connections: shear and moment. A connection may be designed to transfer shear force if the beam is supposed to be hinge connected to the column only. If the beam is rigidly connected to the column, the requirement of a moment connection arises. Shear force is usually involved in moment connections also. Bolts are arranged to be in shear condition to resist shear force in the beam web and tensile and compressive force in the flanges invoked by moment.

Another situation is that the connection is designed to transfer tension from the beam to the column or bracing. The bolts then may be in shear or tensile condition or combined shear and tensile. If prying is intend to happen, it should be taken into consideration.

In all these conditions, usually four members need to be checked for strength or buckling: the column, the beam, the connection member(s) and, of course, the bolts. If shop weld is involved, that is another item to be calculated.

Different connections could be used to fulfill a certain beam-column joint. Sometime it is not obvious which one is most economical. The cost estimation is necessary for this situation. A general idea of cost estimation will be introduce in this paper.

2. Types of bolted beam-column connections

Bolted beam-column connections can be sorted to groups by different circumstances, e.g. by beam and column profiles (I shape or HSS), by bolts working conditions (bearing-type or slip-critical), by forces to be transferred (axial force, shear or moment), by types of connection members (plate or angle), and by methods of connecting (framed or seated). It is reasonable to sort connections into groups according to the force to be transferred and then continue to sort each group into sub-groups by the other factors

mentioned above. Connections transferring axial force are usually used for bracing members which will not be discussed in this paper.

(1). Connections transferring shear force

Two common methods can be used to transfer shear force from beams to columns, framed connections and seated connections.

A. Framed connections

Framed shear connections are designed to transfer shear force by connecting the beam web to the column. Commonly used types are shown in Fig 2.1. , Fig. 2.2 and Fig. 2.3. It can be found in these figures that columns could be I shape or HSS. These are the most commonly used column profiles. The beams are usually I shape. Although HSS profiles may be used as beams, they are usually hard to be bolt-connected to columns. Usually HSS beams are field weld with erection bolts only. In this paper, only I shape beams are considered. Channels may be used as small beams. Most of the connections can still be used for channels.

(A.1). Shear tab beam-column connection

Fig. 2.1 shows some typical shear tab beam-column connections. These connections are widely used to facilitate both fabrication and installation.

(a) is the simplest one. The connection member is only a plate with bolt holes shop welded to the column. The beam web is shop prepared with bolt holes accordingly. With different column profiles and connecting location, the connection could be not exactly the same, as other types shown in Figure 2.1, but the main idea and the design procedure remain similar.

(b) shows beam connecting to web of column. Stiffeners may be required to stiffen the thin web of the column in order to avoid local buckling caused by the eccentric moment.

(c) is different with (a) only of an additional flange cope. The cope is necessary when the beam web height is too tight to arrange a certain number of bolts. With the cope existing, an additional item needs to be checked which will be discussed later.

(d)(e)(f) are connections with HSS columns.

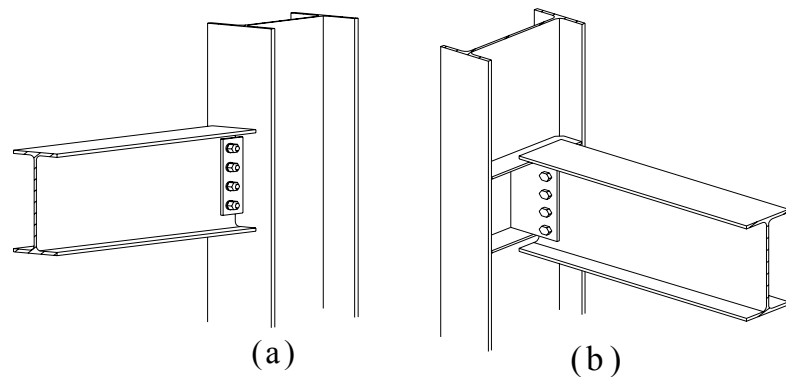
The wall of HSS is usually thin, so the tab plate is usually not welded directly to the wall surface to avoid local buckling, the same reason of (b).

(d) is Tee-type connection. The connection member could be WT profile or built-up Tee (two pieces of plate shop welded Tee shape). The later is more commonly used because of the variety of size. In this type of connection, the Tee is welded at the flange edge close to the corner of the HSS column so that the eccentric moment can be resisted by the walls on the other sides cross the corner. By doing this, the eccentric moment is transferred from weak-axis-bending part to strong-axis-bending parts.

(e) and (f) are through-plate connections. If two sides of beams are connected to HSS column, (f) may be more economical (and/or more reliable) than using two Tee-type connections at each side.

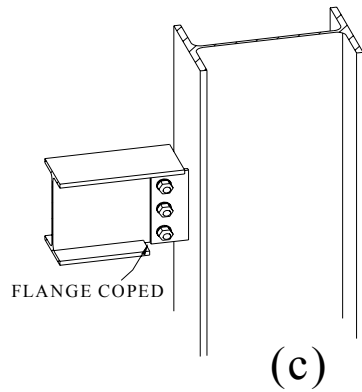
(A.2). End plate beam-column connection

Fig. 2.2 shows typical end plate connections at one side or two sides. An end plate with bolt holes is shop welded to the web of the beam. The column is shop prepared with bolt holes accordingly. It should be noted that if the beams are framed two sides to the column web, and the two end plates are of the same size, at least one end plate should be coped off one bolt to facilitate installation,

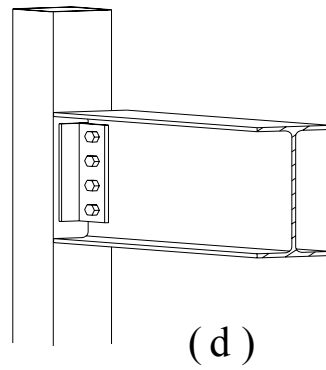


(a)

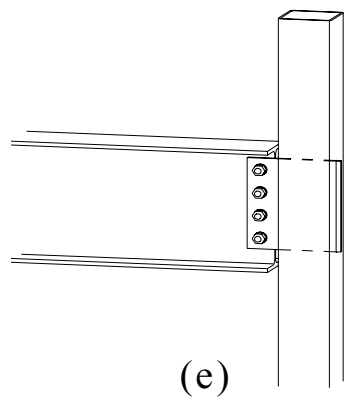
(b)



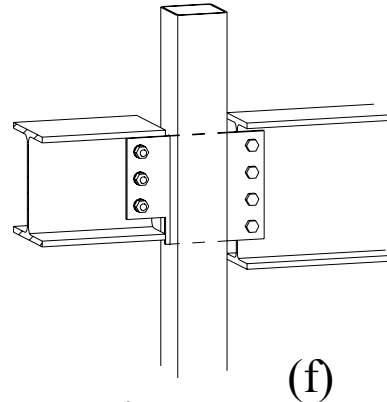
(c)



(d)

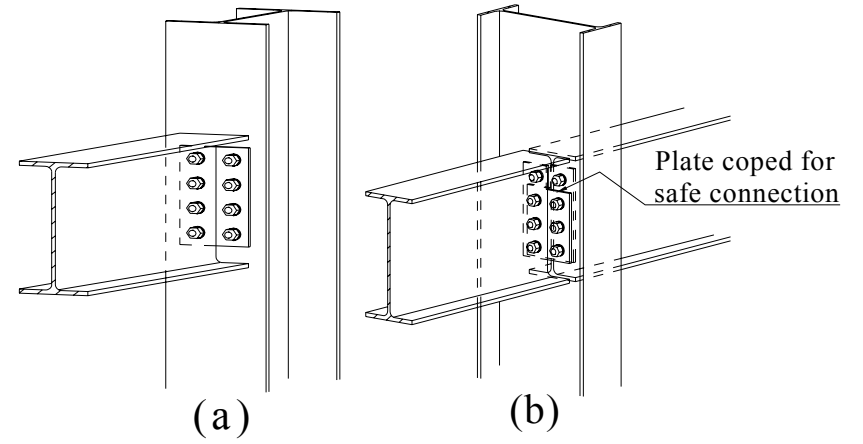


(e)



(f)

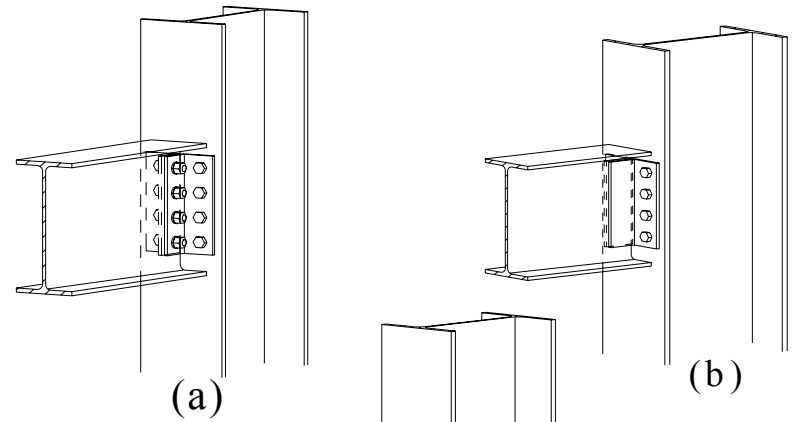
Fig. 2.1 Shear Tab Connection



(a)

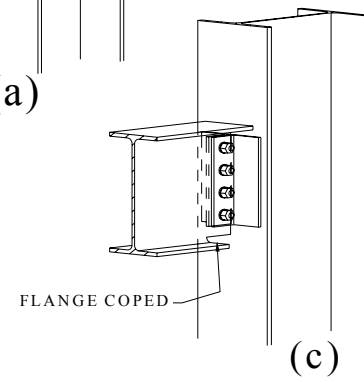
(b)

Fig. 2.2 End Plate Connection



(a)

(b)



(c)

Fig. 2.3 Bolted Angle Connection

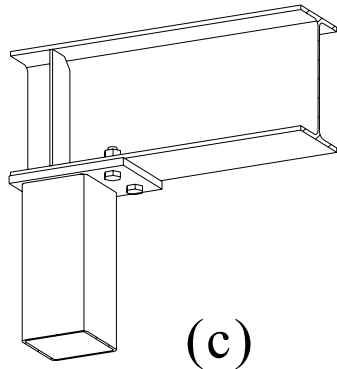
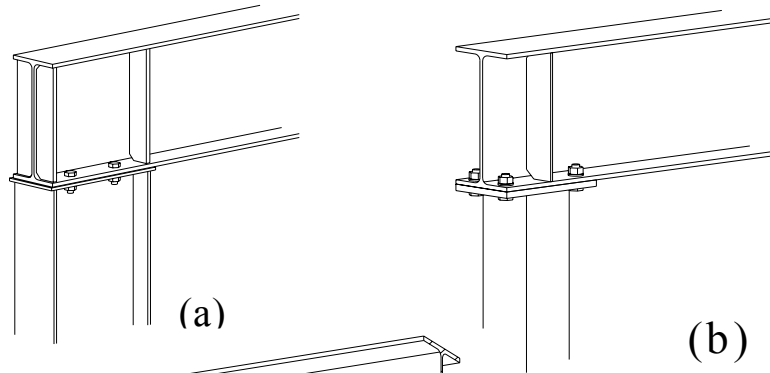


Fig. 2.4 Column Top Seat Connection

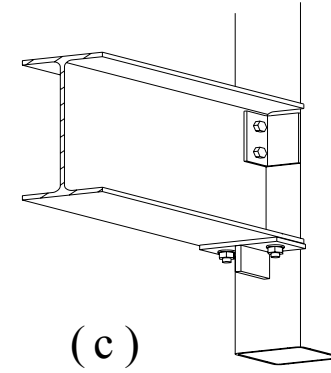


Fig. 2.5 Seat Angle Connection and Built-up Tee-stub Connection

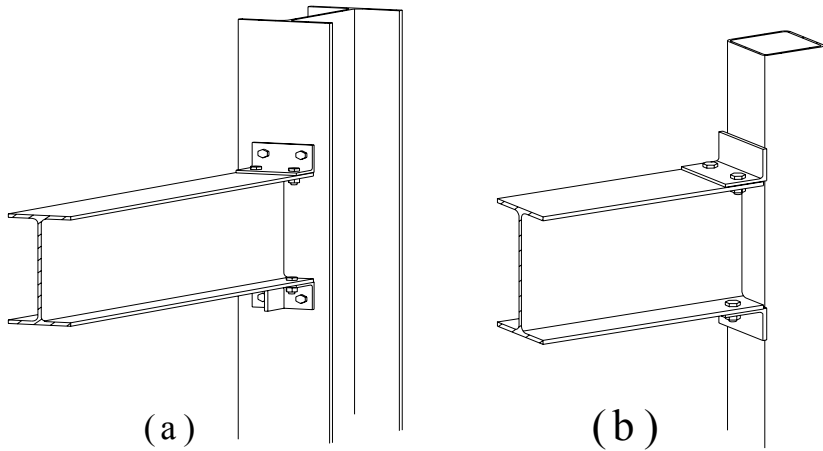


Fig. 2.5 Seat Angle Connection and Built-up Tee-stub Connection

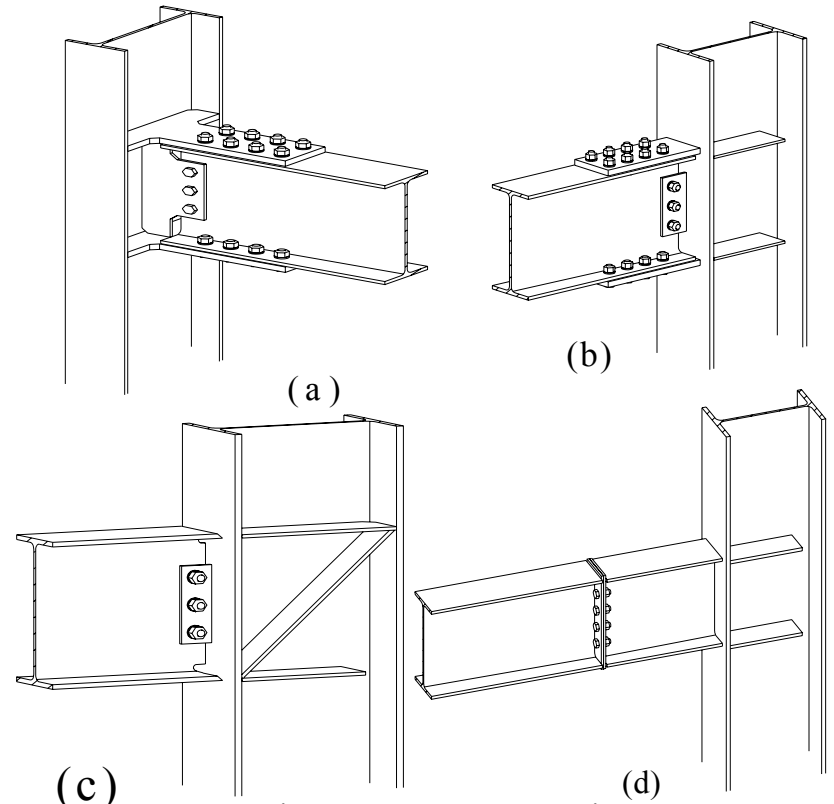


Fig. 2.6 Moment Connection

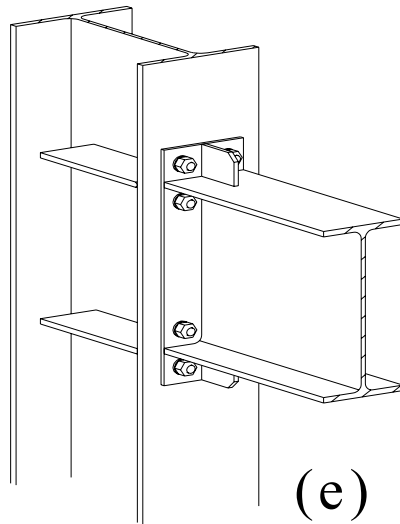


Fig. 2.6 Moment Connection

as shown in (b) of Fig. 2.2..

Comparing with shear tab connections, the end plate connections have advantages as follows:

For both shear tab plate and end plate length within the height of beam web, double number of bolts can be used in end plate connection. That is to say, end plate connection is capable of transferring as twice of shear force as that of shear tab connection. It may not be that much because other parameters may govern. When large shear force needs to be transferred and one row of bolts can not be arranged within the height of the beam web, end plate connection is a good alternation.

Eccentric moment is not involved in end plate connections as that occurs in shear tab connections. So the bolt shear resistance can be fully used to transfer shear force. This is also an advantage for moment sensitive columns.

The disadvantage of end plate connection is its accuracy requirement. The tolerance of bolt connections is only 2mm. A lot

of circumstances will end up with not-matching bolt holes. i.e. the bolt connection can not be fulfilled on site. This may be caused by the mistake of detailer, or shop worker, or site worker. Common reasons are mis-locating of bolt holes in shop or mis-locating of columns on site. These mis-locations are usually very small, but if it is bigger than 2mm or more, it is big enough to kill the bolted connection. Slot holes can be used if applicable for shear tab connections to allow bigger tolerance, while end plate connections do not have this option. If the tiny mis-location happens to a shear tab connection, it can be easily fixed by field welding beam web to the tab plate. For the same problem happening to end plate connections, say, the beam is a little too long or too short, costly modification has to be made.

(A.3). Bolted angle connection

Fig. 2.3 shows three types of double angle connections. Single angle connections are also commonly used to transfer less shear force if applicable.

(a) is all bolted connection. The angles are usually shop bolted to the beam and can be shipped along with the beam to avoid small losing members. The shop bolts are only finger tight which can be easily taken off on site if necessary. If double angles are used, the bolts on the beam web are double sheared (two shear planes in a bolted joint) so that the bolt strength is most efficiently used. This type of connections has the advantages of end plate connections and avoids the disadvantage of them. The price of having these advantages is consuming more material.

Shop weld is usually more economical than using bolts. That is the advantage of (b) and (c) comparing with (a). (b) is similar with end plate connections while the weld is not so dense at the beam web

edge to avoid high stress. The bolts in (c) are double sheared which is an economical way of using bolts as mentioned before. However, the bottom flange of the beam has to be coped as shown to allow the beam to drop in. This cope also reduces the capacity of the beam in the vicinity of the connection which will be discussed later. All these factor may lead (c) to an uneconomical connection.

B. Seated connections

Fig. 2.4, Fig. 2.5 and Fig. 2.7 show some typical beam-column seated connections

(B.1). Beam seated on top of column.

Fig. 2.4 shows connections of beams seated on top of columns. If lateral force and uplift force are not considered in these connections, bolt calculating is not required. The bolts are selected to be compatible with the size of the beam and the column. At the top of the column, the cap plate is provided simply as a practical convenience to assist in the connection between the beam and the column. The length of the cap plate is selected to provide sufficient space for bolting to underside of the beam, and the plate is of nominal thickness. However, when checking the bearing resistance of the beam, if the bearing length is taken as the length of the cap plate instead of the column width, the thickness of the cap plate has to be checked to resist the bending moment caused by the uniformly distributed bearing force transferred from the beam.

(c) is usually used when the column is located against a wall, so the beam is not allowed to stick out too much out of the column.

(B.2). Seat angle connection and built-up tee-stub seated connection

Seat angle connections and built-up tee-stub seated connection are usually used at floor level to connecting beams to the column while the column continues to upper floor as shown in Fig. 2.5.

Although seated beam shear connections are designed to support vertical loads only, eccentricities produced by these connections may be greater than by simple framed shear connections and can influence the design of supporting members.

The seat angle can be stiffened as shown in (a) or unstiffened as shown in (b). The unstiffened angle seat consists of a relatively thick angle either shop-welded or bolted to the column. When the supporting member is a column web, access for welding may be restricted. Load capacity of an unstiffened angle seat is limited by the angle thickness. This capacity can be increased by stiffening the angle; however, stiffened angle seats are more expensive to fabricate, and stiffened seats using tee-stubs built up from plate as shown in (c) are usually more economical.

The seated beam must be stabilized laterally with a flexible clip angle attached either to the top flange of the beam as shown in (a) and (b) or to the beam web near the top of the beam as shown in (c). The clip angle must be thin enough to permit end rotation of the beam. Either welds or bolts can be used to connect the clip angle to the beam and the column. When welds are used, the fillet welds should be located along the toes of the angle.

(2). Connections transferring moment

When a connection is designed to transfer moment, the flanges of the beam have to be connected to the column. The moment is transferred by the pair of tension and compression forces in the top and bottom flange of the beam. It should be noted that usually moment connections transfer shear force as well, so the web of the beam is also connected to the column.

Fig. 2.6 shows some commonly used moment connection.

(a) and (b) are connections with bolted flanges and shear tab bolted at the beam web. The beam flanges are bolted to plates to transfer tension and compression force which composing moment. The

moment plates are spaced to accommodate rolling tolerances for beam depth and flange tilt, and nominal shims are provided to fill any significant gap. Minor gaps are closed by the action of bolting.

(c) illustrates beam flanges field-welded directly to the column with groove weld. The beam flanges are shop prepared with grooves and rat holes are also prepared on the web to facilitate field welding. Backing bars and run-off tabs for the welds may be required. Shear capacity is developed by web framing plate. Other types of shear transferring methods such as seat angle or web framing angle can also be used. The shear connection also acts as construction support while the field welds are made. Actually this connection should be considered as a welded connection which is beyond this paper.

(d) illustrates the use of short beam sections shop welded to the column, and field-bolted to the beam near a point of contra flexure. An end-plate connection is shown but lapping splice plates for the web may be more economical depending on the forces to be transferred and the relative ease of achieving field fit-up. This connection avoids field welds and also avoids using a lot of bolts and thick plates to transfer the significant tension and compression forces in the flanges. The disadvantage is that the contra flexure point is usually not certain. If the assumed contra flexure point is too far away from the real one, the actual moment will be less than or even over the design value.

(e) shows a heavy plate shop-welded to the end of the beam and field-bolted to the column. The end plate distributes flange forces over a greater length of column web than a fully welded joint, but prying action must be considered. Stiffeners can be used as shown to reduce the prying action.

3. Materials used in bolted beam-column connections

(1). Bolt

There are three kinds of bolts used in steel construction in Canada. These are high-strength structural bolts manufactured under the American Society for Testing and Materials (ASTM) specifications A325 and A490, and common bolts manufactured under ASTM A307. The A325 and A490 bolts are structural bolts and can be used for any building application. A307 bolts, which were referred to previously as common bolts, are also variously called machine bolts, ordinary bolts, and unfinished bolts. The use of these bolts is limited primarily to shear connections in nonfatigue applications. Only high-strength structural bolts A325 and A490 are involved in this paper.

Structural bolts can be installed pretensioned or snug tight. Pretensioned means that the bolt is torqued until a tension force approximately equal to 70 percent of its tensile strength is produced in the bolt. Snug-tight is the condition that exists when all plies are in contact. It can be attained by a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench. Pretensioned bolts are usually used in slip-critical connections or when the bolts are carrying tensile cyclic loading. Refer to [1]§22.2.2 for requirement where pretensioned bolts should be used.

Structural bolts are available in many sizes in both imperial and metric series. The bolt diameter ranges from 1/2" to 1-1/2" in imperial series and from 16mm to 36mm in metric series. Most standard connection tables, however, apply primarily to 3/4" and 7/8" imperial bolts and 20mm and 22mm metric bolts. Shop and erection equipment is generally set up for these sizes, and workers are familiar with them.

Different types of washers are required depending on the specified conditions. Refer to [1]§23.5 for detailed requirements.

(2). Plate and Angle

Plate and angle are the most commonly used materials for bolted connections. The plate and angle should provide adequate flexibility for group of bolts to work together. i.e. potential for minor bolt hole deformation is required. For this reason, the maximum thickness of the material is limited to a certain level. Refer to connection design tables in [2] for detailed information. High strength material should not be used, for the same reason. Common grade of plate and angle is 300W.

4. Design of bolted beam-column connections

The general procedure of bolted beam-column connection design is:

- a. Determine the factored load to be transferred by the connection. Different load combinations should be checked according to National Building Code of Canada (NBCC) 1995-§4.1.3 to find out the most severe effect.
- b. Choose connection type

The parameters to be taken into consideration are:

Load to be transferred.

Economy (Material, Shop cost, Erection cost, Time consuming, Climate, Weather, etc.)

Beam and column profiles.

Location.

Aesthetics.

Material availability.

etc.

- c. Choose bolt grade, size and connection part grade and size.
- d. Calculate number of bolt required.
- e. Check connection part capacity (bearing, tension, compression, shearing, buckling, etc.).
- f. Calculate shop welds.
- g. Check beam and column capacity at connection (bearing, buckling etc, in most case, stiffener design)

Calculations involved in a bolted connection design are discussed below.

(1). Bolt capacity

Bolt capacity of force resistance needs to be calculated in most of the connections except some seated connections as shown in Fig.2.4 and Fig.2.5 (bolts on beam flange).

(a). Bolts in Shear

This is the most common working condition of bolts. This item needs to be checked in the following sample connections:

All of Fig.2.1, Fig.2.2, Fig.2.3, Fig.2.6 (a)~(d)

Code citation

§13.12.1 Bolts in Bearing-Type Connections

For bolts subject to shear or tension, ϕb shall be taken as 0.80.

§13.12.1.1 Bolts in Shear

The factored resistance developed by a bolted joint subjected to shear shall be taken as the lesser of

- (a) the factored bearing resistance, B_r , given in Clause 13.10(c); or
- (b) the factored shear resistance of the bolts, which shall be taken as

$$V_r = 0.60 \phi_b n m A_b F_u$$

When the bolt threads are intercepted by any shear plane, the factored shear resistance of any joint shall be taken as 0.70 of V_r .

For lap splices with $L \geq 15d$, where d is the bolt diameter and L is the joint length between centers of end fasteners. The shearing resistance of the bolts shall be taken as $1.075-0.005L/d$, but not less than 0.75 of the aforementioned values.

\$13.10(c) Load bearing in bolted connections

$$B_r = 3 \phi_{br} t d n F_u$$

where ϕ_{br} = taken as 0.67

F_u = tensile strength of the connected material

[1] no longer implies that threads are excluded from the shear plane when the material thickness adjacent to the nut is equal to 10mm or bigger. Without special precautions, thickness less than 10mm may allow threads to be intercepted. For practical reasons, it is suggested that bearing-type shear connections be designed on the assumption of intercepted threads when combinations of thin material and detailing for minimum bolt stick-through (the nuts) are expected.

Special condition of shear tab connections

Astaneh identified that the strength of shear tabs is a function of several variables. The first is the stiffness of the supporting member. Shear tabs on flexible supports have larger bolt eccentricity, and therefore lower resistances, than do those on rigid supports. The eccentricities are also a function of the number of bolts in the connection. For rigid supports, such as columns, efficiency in terms of capacity per bolt is a maximum for four bolts because the effective eccentricity is zero.

Resistance of shear tab connections using common bolts can be found in Table 3-41 in [2].

(b). Bolts in Tension

Code citation

\$13.12.1.2 Bolts in Tension

The factored tensile resistance developed by a bolt in a joint subjected to tensile force, T_r , shall be taken as

$$T_r = 0.75 \phi_b A_b F_u$$

The calculated tensile force, T_f , is independent of the pretension and shall be taken as the sum of the external load plus any tension caused by prying action.

Detailed explanation of prying action and design tables can be found in [2] Section 3, bolts in tension and prying action.

(c). Bolts in Combined Shear and Tension.

Fig. 2.6(e) is a typical connection in which bolts are in combined shear and tension. The total 8 bolts resist shear force and the 4 bolts adjacent to the top flange of the beam also arise tensile force to resist moment.

When bolts are supposed to resist eccentric moment, like the bolts on column flange in Fig. 2.3(a), combined shear and tension also need to be checked.

Code citation

\$13.12.1.3 Bolts in Combined Shear and Tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

Where V_r is as given in Clause 13.12.1.1 and T_r is as given in Clause 13.12.1.2

(a) Bolts in Slip-Critical Connections.

Refer to [1] §13.12.2 for capacity of bolts in slip-critical connections.

(2). Weld capacity

The main types of welds used for structural steel are fillet, groove, plug, and slot. The most commonly used weld is the fillet. For light loads, it is the most economical, because little preparation of material is required. For heavy loads, groove welds are the most efficient, because the full strength of the base metal can be obtained easily. Use of plug and slot welds generally is limited to special conditions where fillet or groove welds are not practical. Shop welds are usually more economical than bolts. In bolted beam-column connections, the connecting parts (plates or angles) are usually shop welded to one member and field bolted to the other one to be connected.

Fillet welds should be used in all the sample connections shown in Fig.2.1 through Fig. 2.5. Groove welds should be used in the moment connections at beam flanges locations as shown in Fig. 2.6. to obtain full strength of the beam.

Only three types of working conditions of the shop welds are involved in the sample connections, these are:

Fillet welds for shear. Groove welds for tension and compression normal to axis of weld.

Code citation

§13.13.1 Welds general

The resistance factor, ϕ_w , for welded connections shall be taken as 0.67.

§13.13.2.2 Fillet welds

The factored resistance for shear and tension or compression-induced shear shall be taken as the lesser of

(a) for the base metal

$$V_r = 0.67 \phi_w A_m F_u$$

(b) for the weld metal

$$V_r = 0.67 \phi_w A_w X_u (1.00 + 0.50 \sin \theta^{1.5})$$

Where

θ = angle of axis of weld with the line of action of force (0° for a longitudinal weld and 90° for a transverse weld).

A_m = shear area of effective fusion face

A_w = area of effective weld throat.

§13.13.3 Tension Normal to Axis of Weld

§13.13.3.1 Complete Joint Penetration Groove Weld Made with Matching Electrodes

The factored tensile resistance shall be taken as that of the base metal.

§13.13.4 Compression Normal to Axis of Weld

§13.13.4.1 Complete and Partial Joint Penetration Groove Weld, Made with Matching Electrodes

The compressive resistance shall be taken as that of the effective area of base metal in the joint. For partial joint penetration groove

welds, the effective area in compression is the nominal area of the fusion face normal to the compression plus the area of the base metal fitted in contact bearing.

According to §13.13.3.1 and §13.13.4.1, in a moment connection, if a part is connected by complete joint penetration groove welds made with matching electrodes, the welds do not need to be calculated. Complete joint penetration groove welds are commonly used in moment connections.

(3). Connecting part capacity

(a). Framed shear connection

Possible failure mechanisms of connecting parts in framed shear connections are shown in Fig. 4.1.

The end shearing failure as shown in (b) is that the plate be sheared out between the end fastener and the end of the plate. This failure

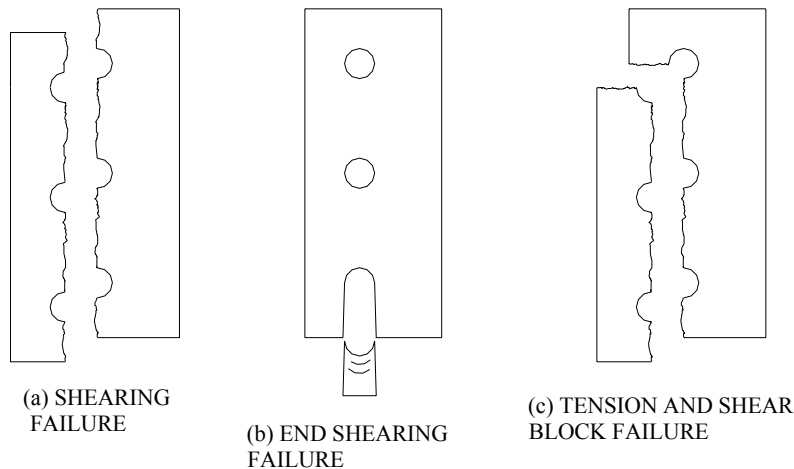


Fig. 4.1 Failure mechanisms of shear plate or angle mechanism is avoided by limiting the distance between the centerline of the bolt and the end of the plate (end distance).

Refer to [1] §22.3.4 for the requirement of minimum end distance. The resistance of tension and shear block failure as shown in (c) is illustrated in [1] §13.11.

Code citation

§13.11 Tension and Shear Block Failure

The factored resistance of a connected part whose failure mode involves both tensile fracture and either shear yielding or shear fracture shall be taken as

(a) for gusset plates, framing angles, and shear tabs, and the ends of tension members, the lesser of

- (i) $T_r + V_r = \phi A_{nt}F_u + 0.60 \phi A_{gv}F_y$
- (ii) $T_r + V_r = \phi A_{nt}F_u + 0.60 \phi A_{nv}F_u$

(b) for coped beams the lesser of

- (i) $T_r + V_r = 0.50 \phi A_{nt}F_u + 0.60 \phi A_{gv}F_y$
- (ii) $T_r + V_r = 0.50 \phi A_{nt}F_u + 0.60 \phi A_{nv}F_u$

where

A_{nt} = the net area in tension for block failure

A_{gv} = the gross area in shear for block failure

A_{nv} = the net area in shear for block failure

The shearing failure resistance as shown in (a) should be taken from the same formulae as those of tension and shear block failure except that A_{nt} equals to zero.

(b). Seat angle connection and built-up tee-stub seated connection

Commonly used seated connections are unstiffened seat angle as shown in Fig. 2.5(b) and built-up tee-stub seated connection as shown in Fig. 2.5(c).

The capacity of unstiffened seat angle connection is governed by the stiffness of the angle seat and the bearing resistance of the supported beam. Detailed explanation and design tables can be found in [2] Section 3~Seated beam shear connections.

The capacity built-up tee-stub seated connection is governed by the welds which connect the tee-stub to the column. The thickness of the tee-stub plates is required to be not less than the web thickness of the supported beam. The bearing capacity of the beam needs to meet the requirement also. Detailed explanation and design tables can be found in [2] Section 3~Stiffened seated beam connections.

(c). Moment connection

The plates connecting the beam web in moment connections like (a)(b)(d) in Fig. 2.6 are same as those in framed shear connections.

The failure mechanisms of the plates connecting flanges are same as (b) and (c) in Fig. 4.1 plus failures as tension member as shown in Fig. 4.2. Compression is equal in value with tension, usually the plates connecting flanges are of same size and the slenderness ratio of the plate in compression is very small, so usually the compression plate needs not to be checked.

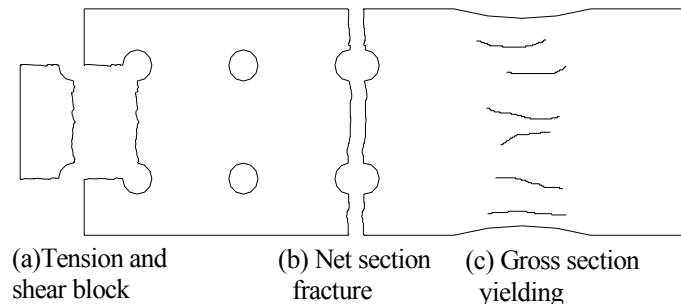


Fig. 4.2 Tension failure

The tension and shear block failure has been discussed before. The capacity to resist the other two failure mechanisms, net section fracture and gross section yielding, are required by [1] §13.2(a)

Code citation

§13.2 Axial Tension Member and Connection Resistance

The factored tensile resistance, T_r , developed by a member subjected to an axial tensile force shall be taken as

the least of

- (i) $T_r = \phi A_g F_y$; (Gross section yielding)
- (ii) $T_r = 0.85 \phi A_n F_u$; (Net section fracture)
- (iii) $T_r = 0.85 \phi A_{ne} F_u$ (Net section fracture considering shear lag)

The effective net area A_{ne} considering shear lag is defined in [1] §12.3.3

The required stiffness of the end plate in (e) of Fig. 2.6 can be calculated using the similar method as the seated connection of (b) or (c) of Fig. 2.5.

(4). Beam and column capacity at connection

With the bolt arrangement meet the requirement of [1] §22, usually only tension and shear block of beam and bearing stiffeners need to be considered for beam and column capacities. The bearing capacities at bolt holes are considered when checking bolt capacities.

When a beam is coped as shown in Fig. 4.3, the tension and shear block capacity is required by [1] §13.11(b) (See Code citation on Page 11). It should be noted that the capacity of tension and shear block of a coped beam is less than that of a framed plate or angle because the tensile strength attainable on the net section of a coped

beam is reduced. This is done because it is observed in tests that the block of material rotates as it resists the load and the tensile stress is not uniform across the length of the net section.

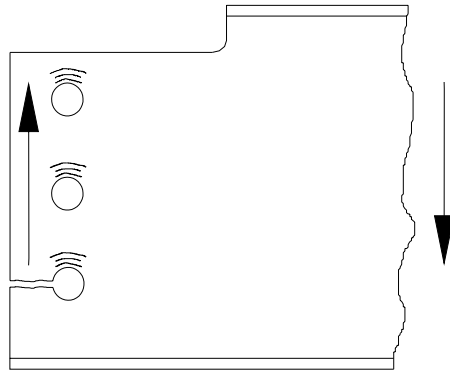


Fig. 4.3 Tension and shear block failure of coped beam

Stiffeners are required for I shaped beam or column when the force acting on the flange exceeds the bear capacity of the web as shown in the sample connections.

Bearing stiffeners may be required in seated connections as shown in Fig. 2.4 and Fig. 2.5.

Code citation

\$14.3.2 Web Crippling and Yielding

The factored bearing resistance of the web shall be taken as follows:

(a) for interior loads (concentrated load applied at a distance from the member end greater than the member depth), the smaller of

$$(i) B_r = \phi_{bi}w(N + 10t)F_y$$

$$(ii) B_r = 1.45\phi_{bi}w^2 \sqrt{F_yE}$$

for end reactions, the smaller of

$$(i) B_r = \phi_{be}w(N + 4t)F_y$$

$$(ii) B_r = 0.60\phi_{be}w^2 \sqrt{F_yE}$$

where

N = length of bearing

$$\phi_{bi} = 0.80$$

$$\phi_{be} = 0.75$$

wherever the bearing resistance of the web is exceeded, bearing stiffeners shall be used (see \$14.4)

\$14.4 Bearing Stiffeners

\$14.4.1

Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the bearing resistance of the web is exceeded (see \$14.3.2). Bearing stiffeners shall also be required at unframed ends of single-web girders having web depth-to-thickness ratios greater than $1100/\sqrt{F_y}$. Box girders may employ diaphragms designed to act as bearing stiffeners.

\$14.4.2

Bearing stiffeners shall bear against the flange or flanges through which they receive their loads and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with \$13.3, assuming that the column section consists of the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at the interior stiffeners, or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, KL , shall be taken as not less than three-fourths of the length of the stiffeners in calculating

the ratio KL/r . Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web to develop the full force required to be carried by the stiffener into the web or vice versa.

Stiffeners for moment connections are required by [1] §21.3

Code citation

§21.3 Restrained Members

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load.

When beams are rigidly framed to the flange of an I-shaped column, stiffeners shall be provided on the column web if the following bearing and tensile resistances of the column are exceeded:

opposite the compression flange of the beam when

$$B_r = \phi_{bi} w_c (t_b + 10t_c) F_{yc} < (M_f / d_b)$$

Except that for members with Class 3 or 4 webs

$$B_r = 640000 \phi_{bi} w_c (t_b + 10t_c) / (h_c / w_c)$$

opposite the tension flange of the beam when

$$T_r = 7\phi t_c^2 F_{yc} < (M_f / d_b)$$

Where

w_c = thickness of column web

t_b = thickness of beam flange

t_c = thickness of column flange

F_{yc} = specified yield point of column

d_b = depth of beam

h_c = clear depth of column web

The stiffener or pair of stiffeners opposite either beam flange shall develop a force equal to

$$F_{st} = (M_f / d_b) - B_r$$

Stiffeners shall also be provided on the web of columns, beams, or girders if V_r calculated from §13.4.1.2 is exceeded, in which case the stiffener or stiffeners shall transfer a shear force equal to

$$V_{st} = V_f - 0.55\phi w_d F_y$$

In all cases, the stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of the column only, the stiffeners need not be longer than one-half of the depth of the column. When an axial tension or compression force is acting on the beam, its effects (additive only) shall be considered in the design of the stiffeners.

5. Connection design example (Bolted end plate connection Formatted Spreadsheet developed)

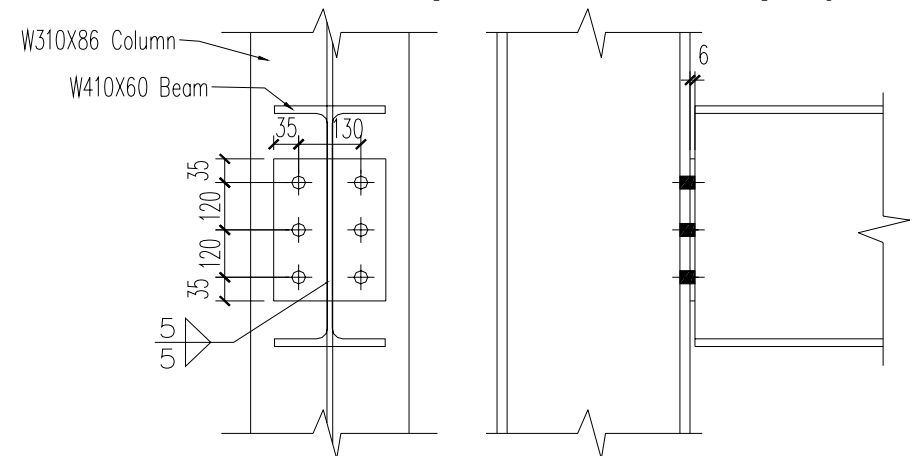


Fig.5.1 Example of End Plate Connection

Fig. 5.1 shows a W410X60 beam to be connected to the flange of a W310X86 column. The factored shear force to be transferred is 450KN. Beam and column are 350W steel and end plate will be 300W steel. A325M bolts and E49xx electrode are available for the connection.

Parameter of column steel section:

Flange thickness $t = 16.3\text{mm}$

Parameter of beam steel section:

Web thickness $w = 7.7\text{mm}$

5.1 Calculate number of bolts required.

From §13.12.1.1: Assume M20 A325 bolt with thread intercepted.

$$n = \frac{V}{0.6 \phi_b m A_b F_u \cdot 0.7} = \frac{450 \cdot 1000}{0.6 \cdot 0.8 \cdot 1 \cdot 314 \cdot 825 \cdot 0.7} = 5.2 \quad \text{---}\$13.12.1.1(\text{b})$$

Take 6 bolts, 2x3

5.2 Check plate bearing resistance at bolt holes.

Use 6mm plate, W310X86 flange thickness is $16.3 > 6$, so 6mm end plate thickness will govern.

$$B_r = 3 \phi_b t d n F_u = 3 \cdot 0.67 \cdot 6 \cdot 20 \cdot 6 \cdot \frac{450}{1000} = 651\text{KN} > 450\text{KN} \quad \text{OK} \quad \text{--}$$

§13.10(c)

5.3 Calculate weld

Fillet weld length required:

According to CSA W59, both end plate and beam web thicknesses are between 6mm~12mm, minimum of 5mm fillet weld size is required.

Try 5mm fillet weld both sides of beam web.

$$L = \frac{V}{0.67 \cdot \phi_w \cdot F_u \cdot 5} = \frac{450 \cdot 1000}{0.67 \cdot 0.67 \cdot 450 \cdot 5 \cdot 2} = 223\text{mm}$$

5.4 Check weld length required according to beam web shear:

$$L = \frac{V}{\phi \cdot w \cdot 0.66 \cdot F_y} = \frac{450 \cdot 1000}{0.9 \cdot 7.7 \cdot 0.66 \cdot 350} = 281\text{mm} \quad \text{----}\$13.4.1.1$$

Plus 2 times weld size at start and end, so at least 291mm end plate length is required.

Plate size and hole locations are shown in Fig 5.1

Horizontal pitch of bolt is taken to be 130 as usual gauge.

5.5 Check bolt pitch and edge distance

Bolt pitch and edge distance are check according to §22.3. OK

5.6 Check end plate shear resistance

$$A_{gv} = 310 \times 6 = 1860\text{mm}^2$$

$$A_{nv} = A_{gv} - 3 \times (20 + 4) \times 6 = 1428\text{mm}^2$$

$$V_r = 0.66 \cdot \phi \cdot A_{gv} \cdot F_y = 0.66 \cdot 0.9 \cdot 1860 \cdot 350 / 1000 = 387\text{KN}$$

$$V_r > V/2 \quad \text{OK}$$

5.7 Check end plate tension and shear block resistance

$$A_{nt} = (35 - 20/2 - 2) \cdot 6 = 138\text{mm}^2$$

$$A_{gv} = (240 + 35) \cdot 6 = 1650\text{mm}^2$$

$$A_{nv} = A_{gv} - (20 + 4) \cdot 2.5 \cdot 6 = 1290\text{mm}^2$$

$$T_r + V_r = \phi A_{nt} F_u + 0.60 \phi A_{gv} F_y = 323\text{KN} \quad \text{---}\$13.11(\text{a})(\text{i})$$

$$T_r + V_r = \phi A_{nt} F_u + 0.60 \phi A_{nv} F_u = 369\text{KN} \quad \text{----}\$13.11(\text{a})(\text{ii})$$

$$323 > V/2 \quad \text{OK}$$

6. Cost estimation of bolted beam-column connections

The cost of bolted connections includes:

Cost of material: connection parts (plate or angle), bolt

Cost of connection parts preparation: cut plate or angle, punch or drill holes

Cost of column and beam preparation: cut, cope, punch or drill holes

Cost of shop weld: material (electrode etc.) and labor

Cost of installation

The costs of detailing drawing and shipping are not included because these items usually do not differ with connection types to be chosen.

The volume of material and labor for different types of connections can be calculated once the forces are known and the type of connection is chosen, while the rates for the items listed above differ with locations, shops, installers, material providers, time etc.

To do a cost estimation of connections, the designer needs to get all the information of these rates according to certain situation. Then put these rates in the formatted spreadsheet, cost can be calculated for different types of connections. An optimal connection can then be chosen.

An example of cost estimation of an end plate connection is provided in the developed the spreadsheet.

7. Conclusion

Bolted beam-column connections are widely used in steel construction. Various connections could be chosen to fulfill a task while they all have advantages and disadvantages. The designer

should consider certain parameters such as cost, shop fabrication, shipping and installation to choose a proper one.

Even for a very simple connection such as shear tab, a lot of items need to be checked. Connection design can be time consuming if done by hand and important checking may be missed which may cause trouble.

Well developed spreadsheets are good tools for connection design and cost estimation.

References

- [1] CAN/CSA S16-01 Limit States Design of Steel Structures
- [2] Handbook of Steel Construction, *Canadian institute of Steel Construction*, Eighth Edition, 2004.
- [3] Kulak, Grondin “Limit States Design in Structural Steel”, Seventh Edition, 2002
- [4] Akbar R. Tamboli “Handbook of Structural Steel Connection Design and Details”, 1999