5 Frame Connections

5.1 GENERAL

The detailing of connections is probably the most important part of structural design, and undoubtedly requires more art and experience to achieve both sound and economical solutions than does member sizing. Portal frame connections were no exception until they began to be standardised with the publication of the Australian Institute of Steel Construction (AISC) "Standardized Structural Connections" manuals in 1985 [1] and 1994 [2]. In 2002, the Australian Steel Institute (ASI) evolved from AISC and the Steel Institute of Australia and then in 2007, ASI began to publish a new series of connections manuals known as the *Connections Series*. By the end of 2009, the *Connections Series* consisted of 1 handbook, 2 sets of design capacity tables and 10 design guides covering both simple and rigid connections. The relevant publications for the rigid connections in portal frames are:

- *Handbook 1: Design of Structural Steel Connections. Background and theory*.[3]
- *Design Capacity Tables for Structural Steel Volume 4: Rigid Connections – Open Sections* [4]
- *Design Guide 1: Bolting in Structural Steel Connections*
- *Design Guide 2: Welding in Structural Steel Connections*
- *Design Guide 10: Bolted Moment End Plate Beam Splice Connections* [5]
- *Design Guide 11: Welded Beam to Column Moment Connections* [6]
- *Design Guide 12: Bolted End Plate to Column Moment Connections* [7]
- *Design Guide 13: Splice Connections*

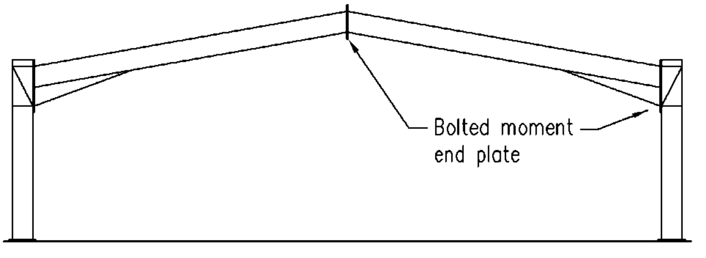

Figure 5.1 *Bolted Moment End Plate Connections at Knee and Ridge*

The most common and economical connections for portal frames consist of bolted moment end plates at the apex and the knee as shown in Figure 5.1. In the past, it was more common to have a shop-welded knee joint and a bolted beam splice consisting of bolted flange and web plates in the rafter at or near the point of contraflexure as shown in Figure 5.2. The advantage of having the bolted splice removed from the knee was that the bolted splice could be designed for a smaller bending moment than the peak bending moment which occurs at the knee. However, although bolted beam splices use less steel than bolted moment end plate splices, they require more hole drilling, more careful fitting, and more handling of heavy beams. The end result is that the combination of the shop welded knee joint and bolted splice is more expensive than the bolted moment end plate at the knee.

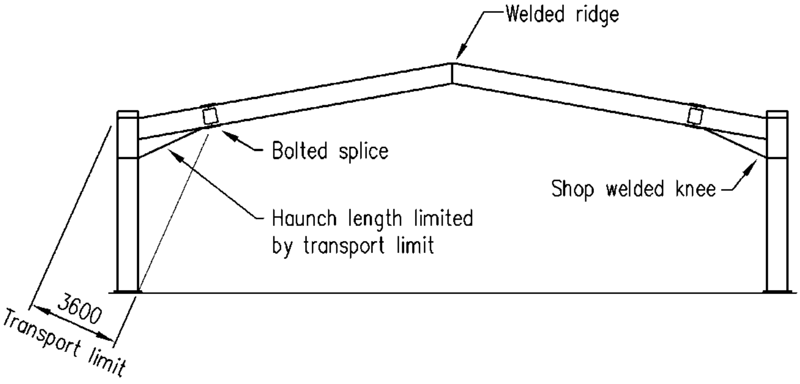

Figure 5.2 *Welded Knee and Bolted Rafter Splice Connections*

5.2 BOLTED KNEE AND RIDGE JOINTS

With bolted moment end plates at the knee as shown in Figure 5.3, the rafter is usually haunched. This has the advantages of not only allowing a reduced rafter size but also reducing the flange and bolt forces at the face of the column because of the extra depth. Standard knee joint details are presented in ASI's Design Capacity Tables Volume 4 [4] and Design Guide 12 [7] for both haunched and unhaunched rafters, while the background theory is presented in *Handbook 1: Design of Structural Steel Connections* [4].

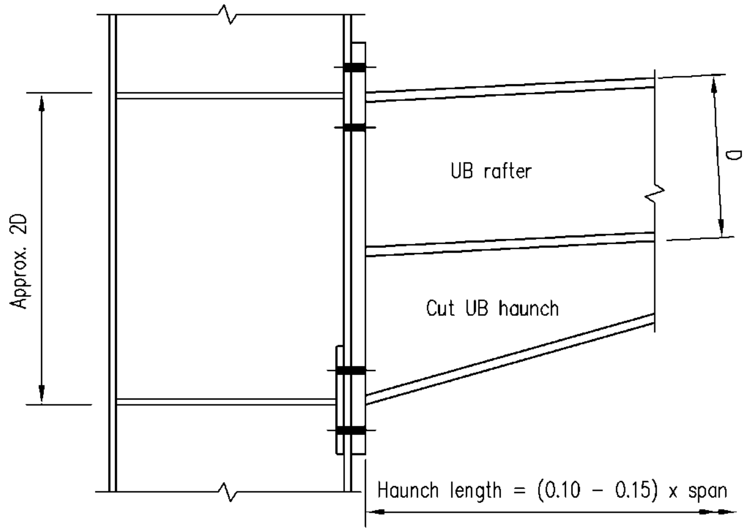

Figure 5.3 *Typical Bolted Knee Joint*

The basis of the design model used in Reference [7] is listed in Section 6 of that reference. Key assumptions include:

- Bolts are fully tensioned in the 8.8/TB category.
- Bolt prying forces are not a consideration because the 'THICK' plate model has been adopted.
- Yield line analysis is used for both the end plates and the column flanges.
- The flanges of the rafter carry the design bending moment and any axial forces.
- The web carries the vertical shear force.
- The shear force is resisted by the bolts on the compression side of the connection.

The column flanges are likely to be thin by comparison with the end plate and will probably require augmentation. For example, UB column flanges are typically between 10 and 16 mm, while a typical end plate is at least 25 mm thick. Hence stiffeners and/or flange doubler plates which are butt welded to the column web may be required.

The most common form of ridge joint is also the bolted moment end plate as shown in Figure 5.4. Compared with the knee joint, the ridge joint is simple to design and fabricate because it consists only of opposing end plates and there is no need for stiffeners or doubler plates.

It would appear that there is a clear advantage in using tensioned Grade 8.8 bolts at the end plates so as to prestress the joint and reduce the tendency of the joint to open (even very slightly) under load. The reduction in joint rigidity due to the use of snug bolts could increase both the frame sidesway movement and the vertical rafter deflections significantly over those obtained from the computer analysis. Tensioned bolts are also the basis of the current ASI connection design models [7] and so snug tight bolts are not recommended.

It is not necessary to nominate these bolts as friction bolts because the prevention of slip of the abutting faces is not critical. The bolts should therefore be designated as 8.8/TB (tensioned and bearing) rather than 8.8/TF (tensioned and friction) so that the fabricator will not leave the abutting faces unpainted. In any case, some surface treatments such as inorganic zinc silicate are accepted as having a friction coefficient at least as high as that for unpainted steel faces.

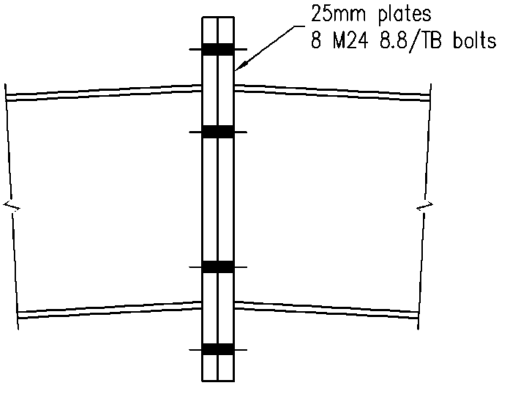

Figure 5.4 *Typical Bolted Ridge Joint*

5.3 COLUMN BASES

5.3.1 Holding Down Bolts

For *pinned bases,* any moment at the base of the column is disregarded and the base is designed for only the axial and shear forces at the base of the column. Two bolts may be sufficient for the applied tension, but the use of four bolts as shown in Figure 5.5, allows riggers to stand and plumb the columns more easily.

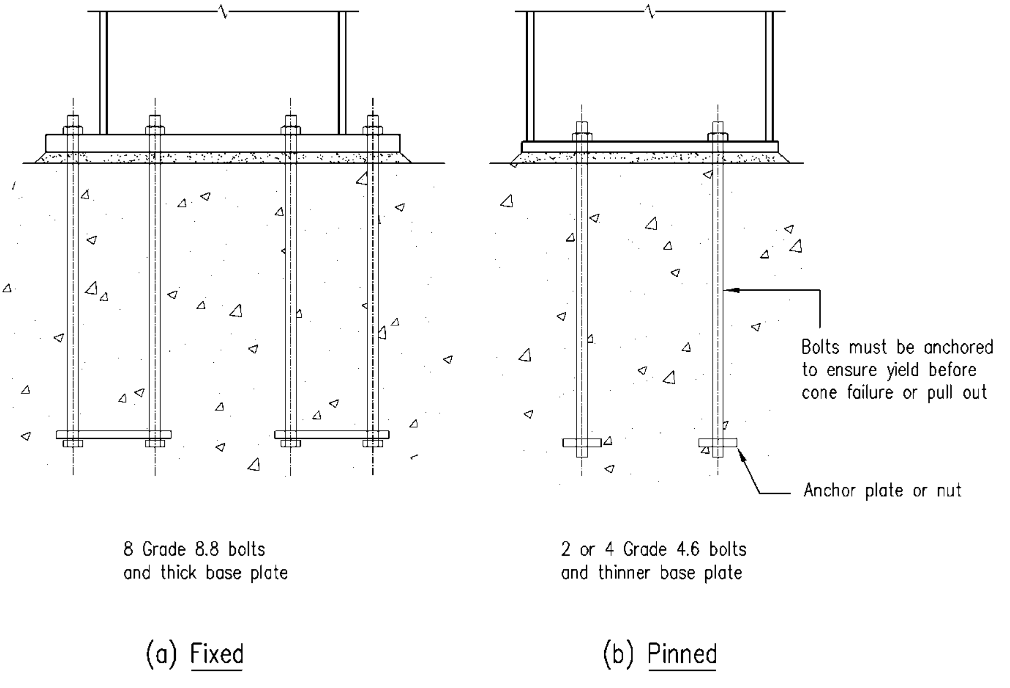

Figure 5.5 *Typical Fixed and Pinned Base Details*

Mild steel Grade 4.6 bolts are preferred because they can be adjusted by bending on site, particularly if there is a sleeve or pocket around the holding down bolt for this purpose. Mild steel bolts can also be tack welded into a cage, whereas Grade 8.8 bolts as shown for *fixed bases* in Figure 5.5a should not be tack welded because welding can have an adverse effect on steel grade in the vicinity of the weld. Regardless of the steel grade, it is recommended that holding down bolts be hot dip galvanised as discussed in Section 7.5.6.

The design of holding down bolts and base plates is addressed in this chapter while the anchorage of holding down bolts in concrete is addressed in Chapter 7. It should be emphasised that holding down bolts the main portal column bases should be anchored so that the bolts will yield before cone failure or pull-out. This is because bending moments will develop at the base of so-called pinned base columns, and there needs to be ductility in the holding down bolt design to cater for bolts to be at their capacity in tension ϕN_{tf}.

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including
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