Economical Structural Steelwork

edited by

John Gardner

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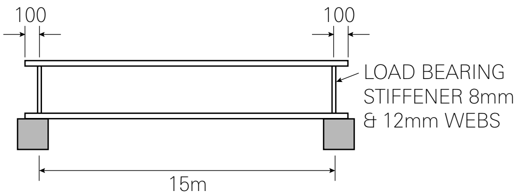
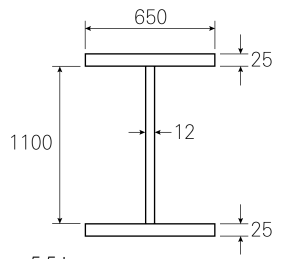
AUSTRALIAN STEEL INSTITUTE

Contents



UNSTIFFENED WEB

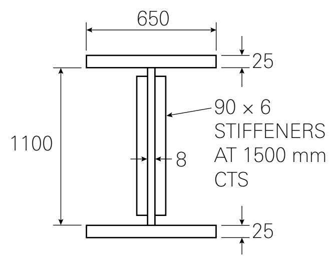


Total Mass	= 5.5 tonnes	
Cost Ratio	= 1.0	
Mass × Cost Ratio	= 5.5	[Cheaper Solution in this case]

INTERMEDIATE STIFFENED WEB

Either:
Stiffeners: 90 × 6 square edge at bars, both sides, at 1500mm centres (18 off)

Or:
Stiffeners: 90 × 6 square edge at bars, one side, at 1500mm centres (9 off)



Total Mass	= 5.0 tonnes
Cost Ratio	= 1.25 for two sided (av); 1.15 for one sided (av)
Mass × Cost Ratio	= 6.3 two sided; 5.7 one sided

The unstiffened web solution is most often the most economic solution but it is not intended to suggest that this is always so.

Each individual situation can be readily assessed by the above process using updated values of the cost ratio for the stiffened web solution.

FIGURE 8.7: Stiffened and unstiffened webs in three plate girders

8.3 Columns

8.3.1 GENERAL

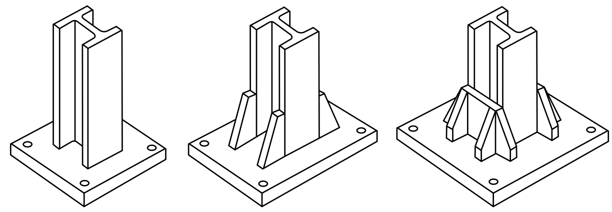
The most economical columns in most building frames will usually be universal beam or column sections. These sections are available in a range of sizes which suit most applications. For applications where good appearance is important, square hollow sections could be considered.

In high-rise buildings it is often economical to consider composite columns, where a relatively small universal column is sufficient to carry dead and construction loads and which, when encased in concrete, becomes a composite column able to carry additional live loads (see Clause 5.5.2).

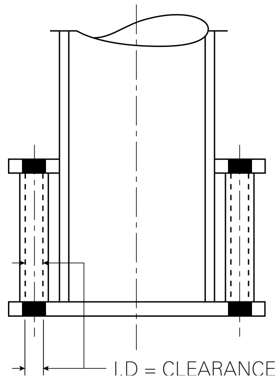
8.3.2 COLUMN BASE PLATES

In the design of column base plates, it is advisable once again to question the wisdom of minimising the mass of material and so introduce extensive fabrication, compared to a heavier base plate simply welded to the column shaft.

Figure 8.8 shows three alternative details for moment resisting base plates.



(a) Slab base plate. (b) Extended ange slab base. (c) Gusseted base plate – avoid, too expensive.



(d) A pipe sleeve allows easy entry of anchor bolts in a double baseplate.

FIGURE 8.8: Column base plate details (moment resisting or fixed)

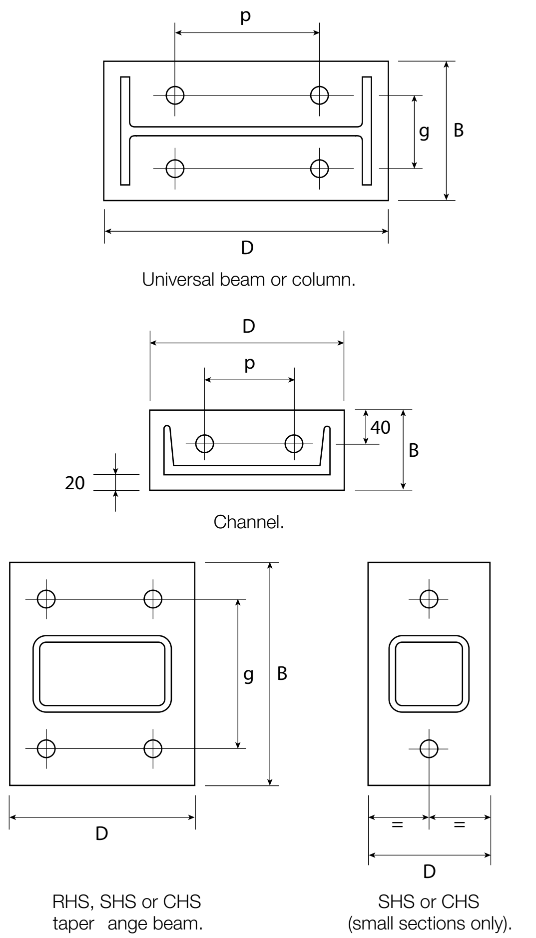
Slab base plate (a) is used widely. It calls for a thicker base plate than the gusseted base plate (c) but requires far less labour for fabrication and therefore it is more economical. Column anges can be extended as shown in (b) to present a larger bearing surface.

Fillet welds should always be preferred for welding the column shaft to the base plate. Only in very rare instances will complete penetration butt welds be required – these should be avoided if possible for maximum economy.

Typical details for pinned base plate connections are shown in Figure 8.9. For the nominally pinned base, there is no need to provide true pin or rocker connections as these are unnecessarily expensive to fabricate. It is recommended that the base plates for main frame columns be of the four-bolt hole type in order to stabilise the columns during the erection stage. Two-bolt hole base plates are satisfactory for secondary columns.

Standardised dimensions for 'pinned base' plates are available in ASI: Connections Design Guides – First Edition 2007 (Ref. 1).

8. Detailing for Economy



Notes:

1. Weld: 6E41 continuous;
2. Bolts: 4.6/S;
3. Column shafts with cold sawn ends provide full bearing contact;
4. All dimensions in millimetres.

FIGURE 8.9: Typical pinned base plates

8.3.3 HOLDING-DOWN BOLTS

One of the greatest problems facing the fabricator/ erector of structural steelwork is inaccuracies in the placing of holding-down bolts. This operation is beyond the fabricator's control and if corrective measures are required on site they usually lead to cost extras and subsequent contractual difflculties.

Several methods have been adopted to overcome this problem and it is essential that the designer presents to the builder very explicit instructions on the method to be used in flxing the bolts. Figure 8.10 shows two typical holding-down bolt details.

In addition to providing flexibility in individual bolt location to ensure matching with base plate drilling, it is good practice to cage bolt groups as shown in Figure 8.11.

Note that bolt cages can only be tack welded to Property Class 4.6 holding down bolts. No welding is permitted to Property Class 8.8 holding down bolts as they are heat treated and welding can alter the physical properties (strength) of the bolts.

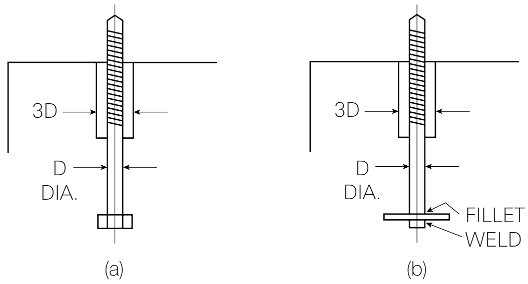


FIGURE 8.10: Holding-down bolt details

8.3.4 COLUMN SPLICES

In high-rise buildings economies can be achieved by running column shafts through three or four floors rather than providing splices at say every second floor (Figure 8.12). Since lengths up to 18m (but see Clause 2.2.3) are now available in most column sections, the greatest economy will be gained in maintaining the same section mass for 3 or 4 floors thus reducing the number of splices required.

Column splices can be welded or bolted. The relative economics of fleld welding should be checked with the fabricator before deciding on adopting this method. Bolted splices will almost always be an economical detail. Figure 8.38 shows typical economic welded splices in columns. Figure 8.39 shows typical economic bolted splices.

It is essential to locate column splices at a convenient level above the floor beams in order to provide comfortable access for the erection personnel to fleld weld or install the bolts (Figure 8.13).

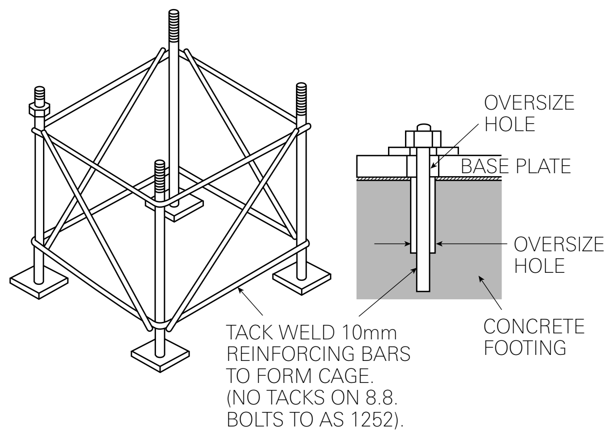


FIGURE 8.11: Typical holding-down bolt cage

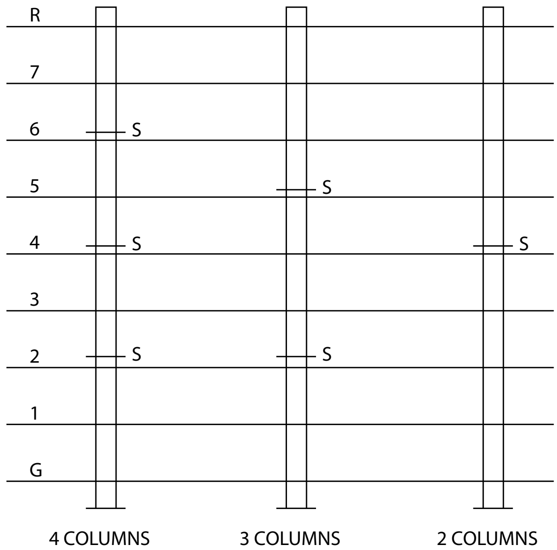


FIGURE 8.12: Minimise number of column splices – 1 is preferable to 3

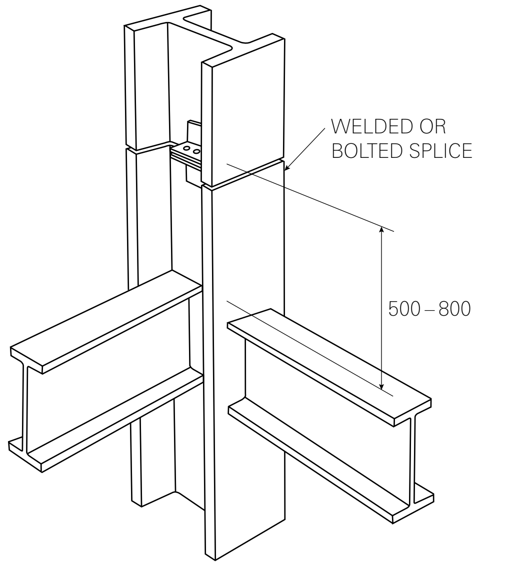


FIGURE 8.13: Preferred column splice locations

8.3.5 COLUMN STIFFENERS

In rigid framed structures, the connections between the beams and columns very often require special stiffening of the column section in order to provide for the satisfactory transfer of forces. These stiffeners add considerably to the fabricated cost of the columns and consideration should be given at the design stage to investigating the alternative use of a heavier column section which requires no stiffening.

The example shows how such an evaluation can be carried out. For the case investigated, it is seen that to increase the size of the column section from a 250UC89 to a 310UC137 is a more economical solution than using the smaller UC with stiffening.

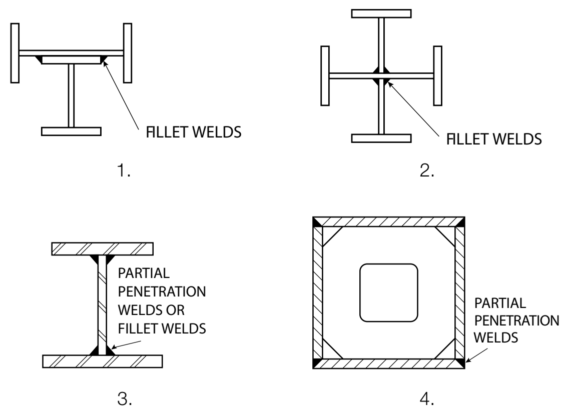


FIGURE 8.14: Economic details for built-up columns in ascending order of fabrication cost

8.3.6 BUILT-UP COLUMNS

Where universal column sections have insufflcient capacity for a particular application, the use of built-up columns has to be considered. Such columns can be fabricated in a variety of shapes. Figure 8.14 shows economic details for built-up columns in ascending order of fabrication cost.

In box columns the detail at the corner can heavily in uence fabrication costs. Where possible the use of fillet welds will afford the best economy – Figure 8.15(a) and (b). Where filet weld sizes required are greater than 12-16mm, partial penetration welds should be considered (Figure 8.15(c)) as a more economic solution. Complete penetration butt welds at corner joints will be rarely required and should only be considered in the vicinity of very heavily loaded rigid beam-to-column connections.

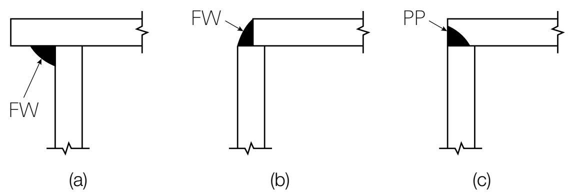
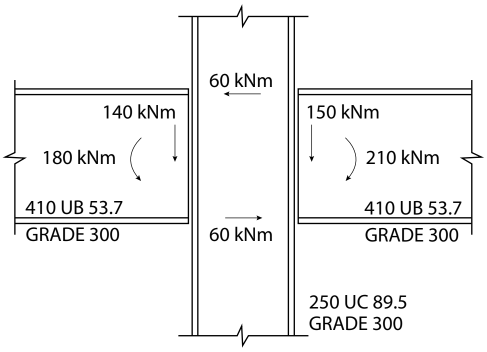


FIGURE 8.15: Welded corner details for box columns

(FW - Fillet Welds PP - Partial Penetration Welds)

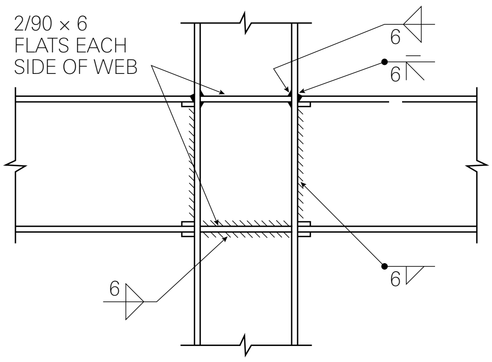
Evaluation of economics of the use of column stiffeners at rigid beam-to-column connection

DESIGN PROBLEM:



SOLUTION 1

Stiffen 250UC89



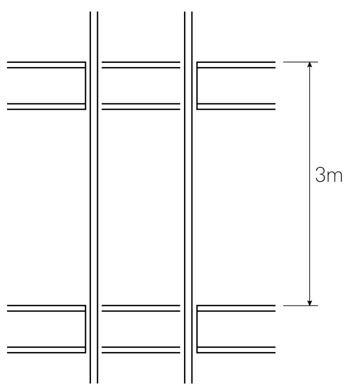
SOLUTION 2

Increase Column Size to Avoid Stiffening

Requires 310UC137 to avoid any column stiffening at all.

Note: 250UC89 = $125 /m
310UC137 = $191 /m
Cost difference = $66 /m

COMPARISON OF SOLUTIONS:



Consider 3m column lift:

Solution 1: Requires 4 stiffeners at $78 = $312

Solution 2: Requires 3m × $66 /m = $198

Solution 2 is the more economic

The use of a heavier column with a thicker web and flange may prove more economic in situations such as that illustrated, especially for short column lifts. Each individual situation can be readily assessed by the above process using updated cost information.

Splices in box columns can be either welded or bolted, but more often than not the welded alternative is selected because a bolted splice is only practicable in large box columns where access can be provided to the inside of the box. A partial penetration welded box column splice can be carried out using the detail shown in Figure 8.16(a). Figure 8.16(b) shows a girder connection to box column – site welded. This connection requires accurate fabrication in the overall length of the girder and may present problems if a considerable run of beams in a line are delivered to site with tolerances in length cumulative. In addition, allowance must be made in column erection for weld shrinkage, since the relatively large weld volume required in heavy girder flanges will cause signiflcant shrinkage in length. Columns must be spread by the shrinkage dimension, as shown in Figure 8.17 and for heavy box columns this can lead to erection difflculty.

Figure 8.16(c) shows a girder-to-column connection which avoids the problems encountered with the direct welded connection shown in Figure 8.16(b). In the case of a girder stub welded to column in the shop, the control of welding procedures and fabrication tolerances generally will lead to a more economic weld and better quality assurance. The subsequent site splicing of the girder to the stub can be either welded or bolted, but the bolted alternative will normally be less costly. In the case of heavy industrial structures using grid flooring however, the bolted flange splice will interfere with this type of flooring, and consideration should be given to welding the splice for such applications.

Figure 8.16(d) shows a bolted girder-to-box column connection. Where flexible connections are used, the angle cleat connection provides good site flt-up. The web cleats are usually loosely shop-bolted to the girder and allow movement for any out-of-tolerance during erection. For box columns, provision must be made in this connection for access to the inside of the column for bolt installation.

Alternatively, where flexible girder-to-box column connections are employed, the web side plate connection will provide about equal economy. The web side plate can be welded to the column face, thus avoiding the problem of internal access.

8. Detailing for Economy

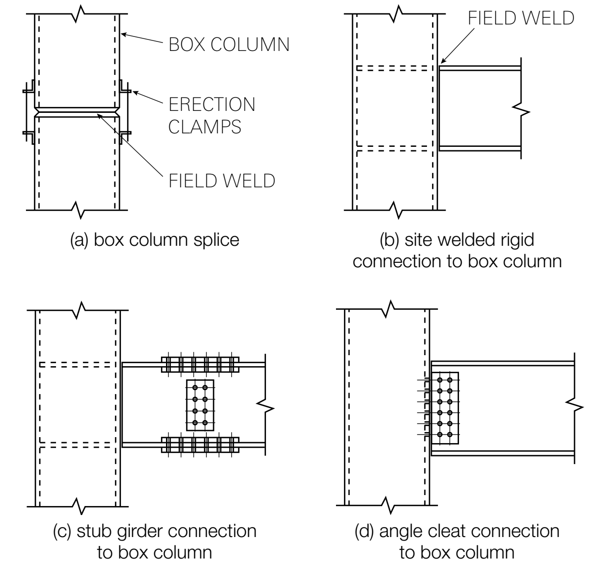


FIGURE 8.16: **Connections to box columns**

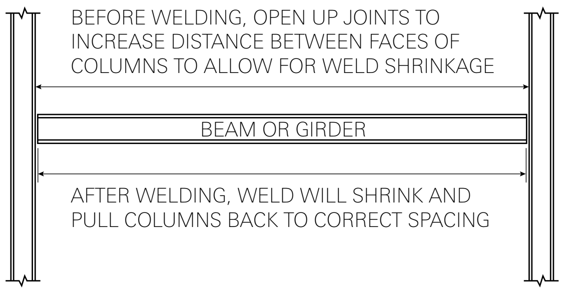


FIGURE 8.17: **Spreading of columns to allow for weld shrinkage**

8.4 Trusses

Welded trusses have in the past provided very efflcient building elements because of the favourable mass/span ratio possible. Although for many industrial building applications, such systems as saw-tooth trusses have been superseded by the portal frame system, there are still many long span applications where truss portals provide an economic solution (see Clause 4.3).

In general, trusses fabricated by welding should preferably use specially developed details suitable for economical welded truss fabrication rather than details borrowed from the days of riveted construction. For too long the old riveted details have been used on welded trusses, on the basis of simply replacing rivets by equivalent welding (see Figure 8.18). This leads to uneconomic fabrication, since it introduces an unnecessary amount of welding and, most importantly, since it requires the truss to be turned during fabrication to weld the angles to the gussets on each side.

Several alternative details offer far more economic welded truss fabrication. Figure 8.19 shows a detail where single angles have been used as both the truss chords and the web members. This provides for the most economic truss fabrication since all welding can be done from one side, thus avoiding turning of the truss during fabrication. Additionally, the gussets have been eliminated by using a long leg angle as a chord member. Obviously this detail requires the designer to consider the eccentricities involved in the design, but it appears in most cases that the use of slightly heavier angles will cater for these eccentricities.

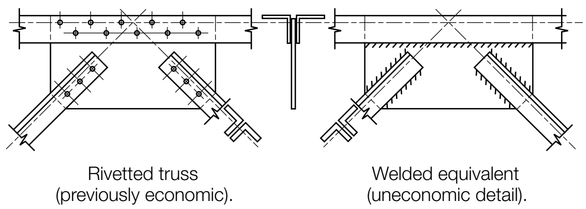


FIGURE 8.18: **Equivalent truss detailing**

Alternatively a T-section can be used for truss chord members with single angle web members welded to the vertical leg of the tee (see Figure 8.20). The T-sections would usually be split universal beam or column sections – an operation that can be economically carried out by most fabricators.

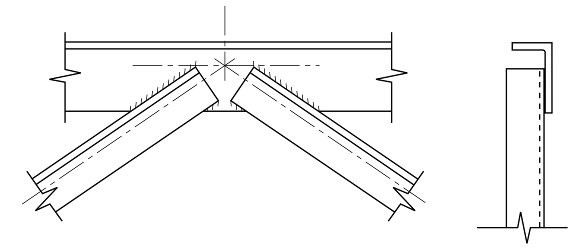


FIGURE 8.19: **Single angle welded truss**

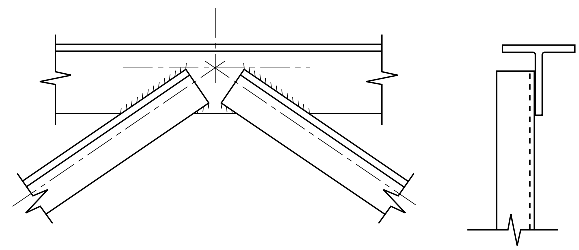


FIGURE 8.20: **Split tee welded truss**

In large heavy trusses, (i.e. those fabricated from universal beam or column sections), care must be taken with detailing to ensure optimum economy. In these cases the detail at the intersection of members can lead to very costly fabrication and it is suggested that the spreading of intersection points can provide a better detail where members can be plain mitre cut to length rather than having double mitre end preparations. The resulting eccentricity can usually be accommodated by the relatively massive chord members in such trusses. Figure 8.21 illustrates the use of universal sections in a welded truss while Figure 8.22 illustrates the use of rectangular hollow sections. In both cases, detail (b) is preferable to detail (a).