8 Beams and Plate Girders

8.1 Introduction

In most structures containing suspended floors the beams or girders represent a significant proportion of the total steel content and for this reason special attention needs to be given to their cost-effective application. Reference has been made in earlier chapters to the efficient functioning of beams and girders in terms of their performance in the overall structure; in this chapter the more detailed aspects of their design, detailing and fabrication are discussed.

8.2 Beam sections

The cheapest form of beam is one made from a standard rolled I-section in the common steel grade, 300WA, and having simple end connections. The great majority of beams used in practice fall within this description.

Obviously, for a given section modulus, the most economical section is the one having the lowest mass per metre. A study of the section tables in the Steel Construction Handbook (Ref. 5) will show that when comparing beams of nearly equal section modulus, it is always the deepest one that has the lowest mass. This affords a simple guide to the selection of a minimum-mass section.

Where a stronger section is required to accommodate high moments caused by large spans or heavy loading, it is always more economical to use a heavier rolled section than to resort to a plate girder or castellated beam, even though the latter sections may offer savings in mass.

If a beam strength greater than that given by the largest available standard rolled section is required, it is better first to try a plated rolled section even though it is considerably heavier than a plate girder. This is because of the vastly more expensive operations involved in cutting the plate girder web and flanges to length, tack welding it on an assembly bed and laying the final welds (which involves turning the girder over). Flange plates welded onto a rolled section can be much more easily handled. They should be made shorter than the beam span and should be arranged for down-hand welding as shown in Fig 8.1, which obviates the need for turning the beam over. If the bottom flange weld is completed before the top weld a camber will be produced in the beam, which may be desirable to counteract loading deflection.
8.3 Castellated beams

By forming a standard rolled I-section into a castellated beam, a section is obtained with an elastic section modulus about 1.55 times that of the rolled section and a moment of inertia about 2.33 times, at no increase in mass. The properties of these sections are given in the Steel Construction Handbook. While these ratios appear very favourable, it must be noted that when section modulus, i.e. moment resistance, governs, it is nearly always cheaper to select a larger plain section than to opt for a castellated section because of the higher cost per metre of the latter. A case for castellation can only be made when a high level of stiffness is required (i.e. a large inertia), or where web openings are needed to allow for the passage of underfloor services in a multi-storey building. An example of where stiffness rather than bending resistance governs is a lightly loaded girder of long span.

It must also be noted that a castellated beam section can only be designed elastically; because of the web holes and the high depth-to-thickness ratio of the web the section cannot behave plastically.

A further point to be considered is that whereas it is desirable that at both ends of the beam a full-depth solid web should be present (to facilitate the fixing of end connections), the pitch of the openings in relation to the beam span will usually not allow this and there is likely to be a web opening at least at one of the ends.

Castellated beams are usually only made in shops specially equipped for their production. The forming of the castellated web shape involves the use of a profile gas-cutter that follows the outline of a specially prepared template, with a different template being required for each serial size of beam. Some steel merchants offer a profile cutting service where they cut the castellated shape in the web, but leave the two halves of the beam attached by small 'stitch' lengths of uncut web. The fabricator then separates the halves, welds them together and trims the ends. Before specifying a castellated beam the designer should ensure that it will in fact be readily available.
8.4 Plate girders

**Cross section**

The most economical plate girder to fabricate is one having minimum mass, equal-sized flanges and no web stiffeners. As with rolled I-sections, for a given section modulus a section with a greater depth will have a lower mass than one with a smaller depth, except in some instances where a thicker web is required in the deeper section.

Where the compression flange is laterally unrestrained, however, it may be necessary to use a wider flange plate to resist the buckling tendency, but this will add to the cost because of the more difficult assembly procedure.

For a uniformly loaded simply-supported girder, which is fully stressed and has a deflection limited to span of 300, the minimum depth/span ratio is about 1:17 for Grade 300W steel. For a girder with a central point load the limit is 1:22. But in practice girders of greater depths than these are usually more economical in material usage.

In order to arrive at a minimum-mass cross section as much as possible of the material should be located in the flanges and as little as possible in the web, consistent with shear requirements. There is usually an advantage, however, in using a somewhat thicker web in order to reduce welding distortion, or to avoid the use of or number of stiffeners. It can be shown that for a given web depth to thickness ratio the minimum-mass cross section is that in which the area of the two flanges combined equals that of the web, i.e. $2A_f = A_w$.

Regarding web slenderness, clause 13.4.1.1 of SABS 0162-1 gives web shear resistances in terms of $h_w/t_w$ ratios and stiffener spacing. According to Clause 13.4.1.3 slenderness ratios up to 277 are allowed for Grade 300W steel, but as already stated the use of very slender webs can cause welding distortion problems.

An important consideration in cost reduction is the use of preferred plate widths and thicknesses for the flange and web elements. All of the girder sections listed in the *Steel Construction Handbook* are made up of plates of preferred thicknesses cut from preferred widths. Each net web depth is equal to a standard plate width (or sub-multiple thereof), less an allowance of 20 mm to 30 mm for edge trimming, the trimming being necessary because of the lack of edge straightness in as-rolled plates. For preferred plate sizes see Chapter 3.

**Web stiffeners**

Where the use of intermediate web stiffeners is considered from the point of view of web stability, it will often be found cheaper to specify a thicker non-stiffened web than to use stiffeners. The cost per ton of a girder with stiffeners on both sides of the web can be higher than that of one without stiffeners by up to 20 per cent or more. Obviously a careful check should be made on the necessity or otherwise of stiffeners because of the potential for saving through their omission.

If it is decided to use intermediate stiffeners, they may be placed on one side of the web only. Although a single stiffener will have to be wider, it will be smaller in section than the
combined width of a pair and obviously the cost of fitting and welding will be very much lower. In addition, the need to turn the girder over to fit the far side stiffeners is obviated.

Where stiffeners are required in any case, for example for the attachment of incoming beams, for the support of top flange loads or on crane gantry girders, advantage should be taken of their presence to use a thinner web. If the spacing of these stiffeners is too great, consideration can be given to inserting intermediate stiffeners between them to allow a thinner web to be specified.

Intermediate stiffeners, and load-bearing stiffeners loaded through the girder top flange, may have their lower ends cut short of the top of the bottom flange; this will simplify both the cutting of the stiffeners and in particular their attachment to the girder. The clearance should be between four and six times the web thickness. Double intermediate stiffeners must have a snug fit against the compression flange of the girder, but need not be welded to it. Single stiffeners should be welded to prevent rotation of the flange. See Fig 8.2.

**Fig 8.2: Plate girder stiffeners**

**Reduction of flange area**

Savings in mass can be made by reducing the size of the flanges towards the ends of a girder. This is more effective in a girder carrying loads concentrated near the centre of the span and less effective when the load is uniformly distributed because of the more rapid reduction in the bending moment towards the ends in the former case. For girders having a length greater than the length of flange plate available, such reduction is justified since splicing of the flange is necessary in any case. For shorter girders, however, reduction would only be warranted in the case of very heavy flanges. It has been suggested that, as a rough guide, splicing should only be adopted if the saving in flange plate material exceeds 100 times the mass of the weld metal involved in butt-welded splices.

In dynamically loaded girders, where a tapered transition is necessary, the reduction of flange section should rather be in the width of the plate than in the thickness, as shown in details (a) and (b) of Fig 8.3 respectively.

**Welding**

Welded plate girders are almost invariably assembled by means of automatic welding machines using the submerged arc process. The most economical flange-to-web weld is the conventional fillet. The girders are laid flat on the assembly bed, that is with their webs horizontal, and the welding machine travels down the length of the girder, laying
welds at the junction of each flange to the web simultaneously. The girder is then turned over and the process repeated on the other side. Fillet welds should always be used, except in the case of dynamically loaded girders, such as crane gantry girders, where complete penetration groove welds should be specified at the top flange; this involves a double edge preparation to the web and is obviously more expensive. The reason for using complete penetration in this application is to avoid fatigue cracking of the weld, which might occur with the double fillet because of the lack of fusion between the flange and the web between the two welds.

In statically loaded girders the web stiffeners should normally be welded to the web intermittently, in this case manually. Continuous welds should only be used where required by the intensity of loading in load-bearing stiffeners, and on all stiffeners in dynamically loaded girders.

8.5 Beam end connections

The cost of end connections makes up a large proportion of the total cost of a beam. For this reason, and because such connections are highly repetitive, standardised types have been developed that will meet the great majority of requirements. The most frequently used ones, as applicable to simply-supported or flexible beam ends, are shown in Fig 8.4. Those in details (a) and (b), which employ bolted angle and welded plate cleats, are the most common and are suitable for beam-to-beam and beam-to-column connections. The type shown in detail (c) is used when the beam connects to a web stiffener on the supporting girder, whilst the seating angle shown in (d) may be used as an alternative to (a) or (b) to facilitate erection.

Standardised connections should be used wherever possible, since a high level of repetition of simple connections makes for economy both in material usage and in labour. Tabulated values of factored resistances are given in Tables 7.14 to 7.19 of the Steel Construction Handbook for a large number of standard connections.

Some of the hints given in the Structural Steelwork Detailing Handbook (Ref. 6) are repeated in Section 8.8 below for easy reference.
Chapters 6 and 7 of *Structural Steelwork Connections – Limit States Design* (Ref. 7) contain detailed information on all aspects of the design of practical and economical beam end connections.

The provision of moment-transmitting end connections on beams and girders adds substantially to the cost of these members, in spite of the reduction in section that can be gained. The suggestions made in Chapter 6 on rafter-to-column connections in portal frames apply in general to beam ends transmitting moment. Where possible, the stiffeners to column webs and to extended beam end plates should be avoided (i.e. items A and B of Fig 6.1), as should the extensions to the beam end plates themselves. However, these items should never be omitted without the approval of the designer, as it is necessary for the strength of the column web and the end plates to be checked.

8.7 Splicing of beams and girders

Site splices in simply-supported beams and girders, when required because of excessive length, should be located as close to the ends of the members as possible. The flange splices need then develop less than the full flange strength because of the reduced moment at these points. Friction-grip bolts should be used to prevent slip of the bolts and consequent sagging of the beam. On rolled sections and the smaller plate girders it is often possible to use a plate on the outer face of each flange rather than double plates, even though the bolts are in single instead of double shear.

Splices in continuous beams and girders should be placed at points of zero or reduced moment, but should not be so small as to impair the general stiffness of the member. In beams without continuous lateral support the splices should be placed near to points
where lateral restraint is present. If this is not possible, full-strength splices will have to be used.

Erection can be simplified in certain cases by the use of part-welded, part-bolted splices, as suggested for portal rafters in Chapter 6 (see detail (b) of Fig 6.2). The projection of the welded bottom flange splice plate of the first beam acts as a seating on which to rest the second beam while it is being bolted.

Further aspects of beam and girder splices are discussed in Chapter 9 of *Structural Steelwork Connections – Limit States Design*.

### 8.8 Camber

Beams and girders may be provided with a camber to counteract the deflection caused by dead loading plus permanently applied live loading. Beams that are to form part of composite construction will often be cambered to counter the effects of ponding at the wet concrete stage of construction. Cambering is usually only necessary when the member is exposed to view and where a visually perceptible deflection would be aesthetically unacceptable. Allowable deflections under working loads are limited to a small fraction of the span, say, $1/200$ to $1/350$ for beams in buildings or $1/600$ for crane gantry girders. These small deflections are often quite acceptable visually, so cambering should only be resorted to when really necessary.

**Beams**

Cambering of rolled I-section beams can be achieved either by means of controlled heating and shrinking using a gas-torch, or in a heavy, usually hydraulic, press. Both operations are time-consuming and costly and as it is difficult to control accurately the amount of camber being produced several attempts will usually have to be made on a trial-and-error basis until an approximately correct camber has been reached.

When cambering of long, shallow beams is thought to be necessary, consideration should be given to using a stiffer uncambered section to reduce deflection. Use of a much heavier section can be considered, despite the increase in material cost, in view of the saving on the high labour costs in the cambering operation.

**Plate girders**

A camber is introduced into a plate girder by cutting the web plate to the cambered shape before assembling and welding the girder. This operation also requires a high labour input because of the laborious process of marking the upper and lower edges of the web and gas-cutting the plate to the curved shape.

**Lattice girders**

Large-span lattice girders can have a camber built in by fabricating the tension and compression members a little shorter and longer respectively than their theoretical lengths. This requires a calculation by the detailer of the change in length of each member to provide the required overall camber.
With bolted girders the assembly of the individual members is straightforward since a bolted structure is self-jigging and will automatically take up its required shape. With welded girders, on the other hand, the assembly jig will have to be adjusted to give the necessary sweep to the chords.

8.9 Detailing of beams and girders

The *Structural Steelwork Detailing Handbook* provides comprehensive information on the detailing of beams and welded plate girders and lists suggestions regarding cost savings. These should be incorporated into designs wherever possible.

Fabricating companies should adopt a standard method of dimensioning holes along the length of a beam. Several methods are shown in Fig 8.5 and the one most suited to the workshop's production methods should be chosen. The methods shown in details (a) and (b) employ chain dimensions, whilst details (c) to (e) running or cumulative dimensions are used. In (a) and (c) the dimensions are to the centres of the hole groups, the holes in each group being marked off with a template. The methods indicated in (d) and (e) are applicable in shops equipped with automatic NC drilling or punching lines.
8.10 Summary of beam detailing hints

Some of the cost-saving hints in the Structural Steelwork Detailing Handbook are repeated here, viz.:

- Use standard detailing practice, as given in the Handbook.
- In a given structure make as many beams as reasonably possible identical or nearly identical.
- For any given beam make both ends identical (or opposite hand), so that it can be erected the opposite way round; this simplifies fabrication and facilitates erection.
- Use standard end connections unless there is some overriding reason to do otherwise. Standard connections for simply-supported beams are given in Tables 7.14 to 7.19 of the Steel Construction Handbook.
- Use standard dimensions for beam end connections.
- Use standard bolt pitches and end distances. These are, in mm:

<table>
<thead>
<tr>
<th>Bolt size</th>
<th>Pitch</th>
<th>End distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>M12</td>
<td>50</td>
<td>25</td>
</tr>
<tr>
<td>M16</td>
<td>60</td>
<td>30</td>
</tr>
<tr>
<td>M20</td>
<td>70</td>
<td>35</td>
</tr>
<tr>
<td>M24</td>
<td>80</td>
<td>40</td>
</tr>
</tbody>
</table>

- In the standard bolted double-angle beam connections in Table 7.17, 7.18 and 7.19 of the Steel Construction Handbook the pitch of M20 Grade 8.8 bolts is given as 100 mm. This is a departure from the recommended value of 70 mm given above and is suggested to ensure an adequate depth of the angle cleats.
- Use standard backmarks on angles and channel flanges and gauges on I- and H-section flanges, as given in the Steel Construction Handbook.
- For simply-supported beam end connections use either bolted angles or welded end plates, not both. The choice will depend on the fabricator's preference.
- Use seated beam end connections as shown in detail (d) of Fig 8.4 instead of bolted web angle cleats or welded end plates where this will facilitate erection.
- Only provide camber in beams and girders when really necessary.
Fig 8.5: Alternative methods of dimensioning bolt hole groups