

Scheme Development: Design of portal frames using fabricated welded sections

This document provides information for the initial design of portal frames made of fabricated welded sections for conventional buildings for industrial or commercial use

Contents

1.	Common use of portal frame construction	2
2.	Choice of portal frame	2
3.	Advantages of fabricated welded sections	3
4.	Section sizing	4
5.	Practical cases	5
6.	Materials, fabrication, transport and erection	7
7.	Connections	8

1. Common use of portal frame construction

Portal frames often form the main structure of industrial and commercial buildings, warehouses, station concourses and sometimes sports complexes which require large volumes of space.

For these “light” or “medium-weight” buildings, steel offers a simple and cost-effective solution while allowing daring architectural options.

Portal frames may also be used to stabilise more substantial buildings for heavy industrial use, especially in steelworks and power stations.

2. Choice of portal frame

Several factors influence the design of all portal frames. Designers may select rolled sections, welded members, lattice structures, or structures combining more than one type of member to best satisfy these criteria.

The most important of these criteria are:

- span between columns
- size and type of loading (static, dynamic – typically travelling cranes etc)
- architectural appearance
- free internal height at the eaves
- relative costs of different solutions

In general, welded portal frames are of greatest benefit in the following cases:

- for highly standardised building solutions, where the reduction in the weight of steel and the economies of mass production offset the additional manufacturing cost. Optimised systems for standardised buildings from specialist suppliers are therefore the subject of intense competition.
- for large spans, where the use of rolled profiles is not economic and the use of lattice rafters is not a good option, for example because of their greater depth of construction.
- for large internal forces, where the choice of rolled profiles is either not possible, due to magnitude of the applied loads, or unacceptable because this would result in reduced spacing between frames.

Combined solutions exist; common cases are:

- welded columns with lattice rafters for large spans
- welded columns with rolled section rafters for industrial buildings with small spans where travelling cranes handle heavy loads



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Figure 2.1 General view of a building made of welded portal frames under construction,

3. Advantages of fabricated welded sections

3.1 Fabricated Welded Sections versus rolled sections

Compared with traditional rolled sections for medium spans, welded members allow the sections to be reduced by adjusting them precisely to the internal forces. This is clearly not feasible with a discrete range of rolled sections.

While maintaining an identical external size, web thickness may be reduced in areas with low shear forces and flange thicknesses may be reduced in areas with low bending moments. This significantly reduces weight – at the expense, however, of having to butt weld between the flange and web plates of different thickness.

The use of welded or lattice sections becomes indispensable for large spans and/or heavy loads where commercial rolled profiles are not suitable.

3.2 Fabricated welded sections versus lattice sections

Compared with lattice beams, the use of built up profiles is more aesthetic. Their shallower depth also reduces the volume of the building. In addition, the reduction in the number of joints required and their simplicity, for average spans, generally makes fabricated sections cheaper.

On the other hand, lattice girders may readily accommodate larger ducts and piping within their depth.

3.3 Choice between uniform or tapered members

There are two types of built-up sections.

- Uniform overall depth and width, possibly with varying plate thicknesses. Thickness may vary generally or the changes may be limited to local reinforcement at connections.
- Built up members with a gradually tapering depth and, possible, changes in flange width.

Other types of sections can also be envisaged, including profiles with asymmetric flanges where the compression flange is wider to improve its stability against lateral torsional buckling. It should, however, be noted that asymmetric and varying width sections require special equipment for efficient manufacture

Asymmetric sections and varying width sections are not discussed further in this document.

4. Section sizing

4.1 General

Section sizing is primarily linked to the bending moment distribution produced by a linear-elastic analysis.

In order to optimise the sections, built-up profiles are used whose second moment of area follows the variations in the bending moment diagram as closely as possible, especially in areas with high bending moments, by:

- increasing the web depth
- increasing the flange thickness
- combining these two effects.

For sections that are subjected to static loading, the sections are initially sized by assuming:

- the flanges resist the bending moment
- the web resists the shear force.

4.2 Dimensions / proportions of sections

Ignoring architectural and aesthetic considerations, it is wise to design welded beams with a greater depth than rolled sections, in order to minimise the flange sizes for a given bending moment.

Similarly, for a required flange cross-section, the flanges should be as wide as possible so that they have optimal resistance to lateral torsional buckling.

The web essentially withstands the shear force. Its depth is maximised to reduce flange sizes for a given bending moment. For a given cross-section, it will therefore be as slender as possible, with a reduced shear strength from buckling.

The whole problem of optimally sizing welded members therefore requires the best compromise between the external dimensions of the sections (depth and width) and the slenderness of the constituent plates, while considering the buckling phenomena of the whole

structure (most often controlled by the overall building lay-out) and local phenomena (the stiffening required for high localised loads, etc.).

4.3 Classification of sections

Experience has shown that greatest economy for portal frames with welded sections is achieved by using elastic theory, without taking account of plastic redistribution, and without even referring to plastic resistance. The following criteria are therefore used:

- Class 3 flanges are chosen so that the compression flanges may reach their elastic limit.
- Class 4 webs are chosen in order to reduce the structure's weight. Shear forces are generally low and may be resisted by such sections in their post buckled condition. Localised loads need special consideration; as far as possible, web stiffeners should generally be avoided although they may be necessary for heavy localised loads.

4.4 Specific types of buckling associated with fabricated welded sections

While the welded profiles significantly reduce the weight of structures, it is important to note that these sections have a considerably higher ratio of $I_y : I_z$ than rolled sections. This type of section is therefore much more sensitive to out-of-plane buckling phenomena. This should influence the construction lay-out.

For example, it may be necessary to prevent the lateral buckling of the compression flange at each purlin, and not at every second purlin, as might have been appropriate if traditional rolled profiles were used.

5. Practical cases

5.1 Typical proportions

To assist in initial design, the following proportions are suggested:

The common depth h of welded members is in the order of $L/30$ for rafters (where L is the span) and $H/10$ for columns (where H is the column height).

The width of the flanges b is usually in proportion with the depth, h . That is:

$$h/5 \leq b \leq h/2$$

The web thickness is between $h/150$ and $h/100$, the greater thicknesses primarily being used at the beam-to-column connection (haunch) where the increase in shear force is linked to the high bending moment gradient.

It should be noted that even greater web slenderness may also be used, but this requires more in-depth study of the buckling phenomena, more precise manufacture and certain precautions during the handling stages when lifting these slender sections with very slender plates.

Example of a common case:

Typical dimensions of members of a 25 m span portal could thus be as follows:

Span	$L = 25$ m
Web depth	$h = 800$ mm
Web thickness:	6 mm thick at central part and 8 mm thick at haunch.
Flange width:	$b = 200$ mm
Flange thickness:	10 mm thick

The web to flange fillet welds would commonly have a minimum throat of 3 to 4 mm.

5.2 Size ranges

To assist in initial design, some limiting dimensions are given below:

Table 5.1 Maximum width for a Class 3 flange, depending on its thickness and steel grade

Thickness (mm)	Maximum width (mm) for steel grade			
	S235	S275	S355	S460
8	225	205	180	160
10	280	255	225	200
12	335	310	270	240
15	420	380	340	300
18	500	460	410	360
20	560	510	450	400

Table 5.2 Web depth in mm, depending on thickness and slenderness

Thickness (mm)	Web depth (mm) for depth/thickness ratio			
	$h_w / t_w = 100$ (*)	$h_w / t_w = 120$ (**)	$h_w / t_w = 140$	$h_w / t_w = 160$
6	600	720	840	960
8	800	960	1120	1280
10	1000	1200	1400	1600
12	1200	1440	1680	1920

(*) the slenderness limit in bending for class 3 web for $f_y = 355$ N/mm² is $h_w / t_w = 100,4$

(**) the slenderness limit in bending for class 3 web for $f_y = 235$ N/mm² is $h_w / t_w = 124$

6. Materials, fabrication, transport and erection

6.1 Steel grades

The choice of steel grade depends on a wide range of criteria, primarily on the loading and the deformation limits.

As a general rule:

- when the deformation criteria are tight (for example, limiting horizontal displacements at column heads for correct operation of travelling cranes), it is wise to opt for a high second moment of inertia which, favours S235 or S275 grade steel rather than S355.
- in the case of heavy loads, no special deflection criteria and where the design is such that the buckling phenomena are controlled, it is wise to select the S355 grade.

It may also be appropriate to make hybrid sections combining the two grades to optimise cost.

Welded sections could therefore use:

- flanges in S355
- web in S235 or S275

6.2 Manufacture of sections

The manufacture of welded profiles requires the procurement of standard width plates, cut to the required width, from specialised wholesalers, or the cutting of these plates in workshops (flame cutting – laser cutting, etc.) from larger sized sheets in order to minimise off-cuts.

Choice depends on the means of production in the workshops and the price differential between procuring plates with the required finished sizes and the cutting of these plates from sheets, requiring additional handling operations.

Straight profiles

Traditional welded profiles are made up of a web and two, usually identical, flanges in order to obtain a doubly symmetric profile.

Each flange is welded to the web using two fillet welds sized to resist the longitudinal shear.

This operation is generally done in three stages:

- the web is laid flat, the two flanges are placed on either side and a continuous fillet weld is applied
- the section is turned over
- the two other fillet welds are applied

In the case of static loading, it is possible to attach the flange and web using a single continuous fillet weld in the mid-span region, away from connections and high shear forces. Near attachments and when required to resist longitudinal shear, the section is turned over and double sided welding is applied.

Single-sided welding is not appropriate where there is dynamic loading

Profiles with a curved radius or with a change in direction

For tapered members where a flange is curved, the change in direction of this flange causes either tension or compression across the web-flange junction. This must be taken into account when sizing these fillet welds, or stiffeners should be added, if necessary.

6.3 Handling: Workshop, in transit and during erection

For very slender and large span members, it is important to carefully study the transport and handling conditions. It is particularly important to prevent the member from twisting under its own weight during these lifting operations; if it does, it may fail in bending/buckling about its weak axis.

7. Connections

7.1 Haunches and ridge joints

Haunches and ridge joints are important details whose design must be carefully considered. They are generally made using endplates and bolts in tension.

In the case of portals using built-up profiles, increased resistance and stiffness of the rafter-to-column connection can easily be obtained by increasing the depth of the splice section, and also increasing the web thickness in this area of high shear force, as shown in Figure 7.1 and Figure 7.2. Stiffening may thus be eliminated or minimised. By contrast, rolled profiles would have required a haunch and the possible addition of web stiffeners.

Design using fabricated sections thus responds to these specific design points both in terms of aesthetics and cost.

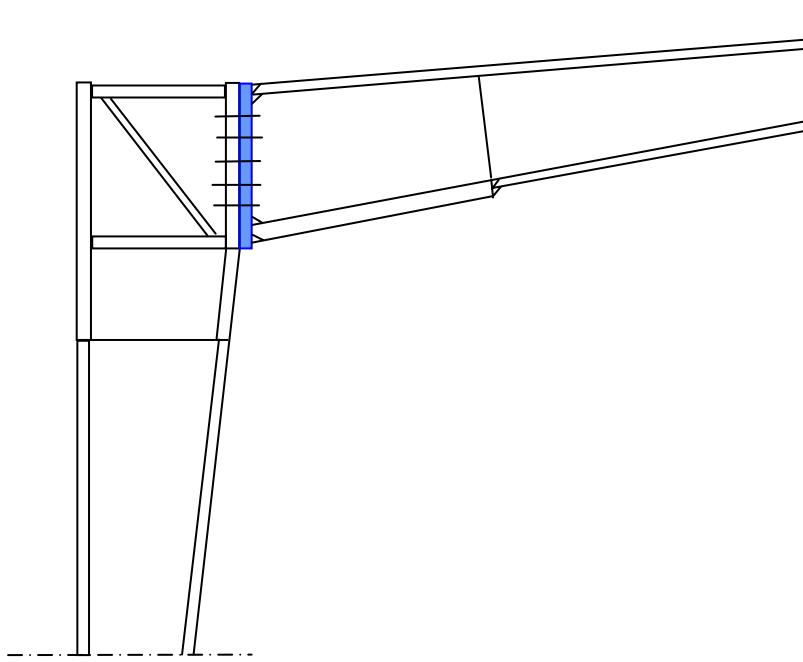


Figure 7.1 Schematic of a portal haunch

7.2 Splices

For large spans and heavy construction, in order not to have to use special transport, which is always costly and requires special authorisation, it is often necessary to split the rafters/columns into several lengths, with splices being made on site.

It might also be more economical to extend the column using a rafter stump in order to move the site connection to an area with lower internal forces. (However low the moment, this connection must still be rigid in order not to violate the assumptions of the elastic global analysis). However, depending on the size of the stump, this design may prove unacceptable for transport.



Photo: Jean-Pierre Muzeau – Copyright APK

Figure 7.2 *Column-rafter joint*

7.3 Column bases

Column bases are also important for the overall design.

It is useful to remember that the sizing of the whole structure, and the behaviour of portal frames vis-à-vis horizontal deformation, in particular, is strongly dependent on the restraint conditions of the column bases in the foundations.

As a general rule:

- ❑ Frames with rigid column bases achieve substantial savings in the weight of the superstructure while minimising horizontal deformations. This solution is therefore recommended for portal structures supporting travelling cranes, where there are severe deformation limitations.

- ❑ This solution is also suitable for structures carrying heavy loads, or highly seismic conditions, or cyclonic winds. It is most effective in reducing column size.
- ❑ However, the major disadvantage is that this solution requires substantially larger foundations. They have to be sized to resist the overturning moment. The column base connection requires large holding down bolts and often requires considerable stiffening to the column, as shown in Figure 7.3.

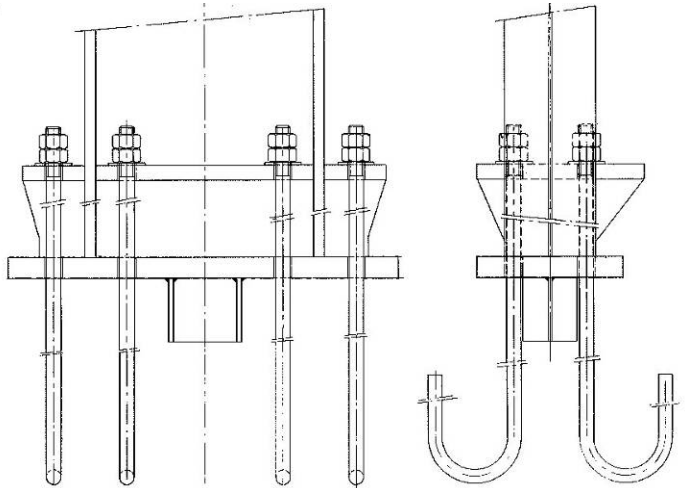


Figure 7.3 Rigid column base

- ❑ Pinned column bases are more commonly used, especially for light or medium steelwork. Figure 7.4, Figure 7.5 and Figure 7.6 show different solutions based on different national practices.

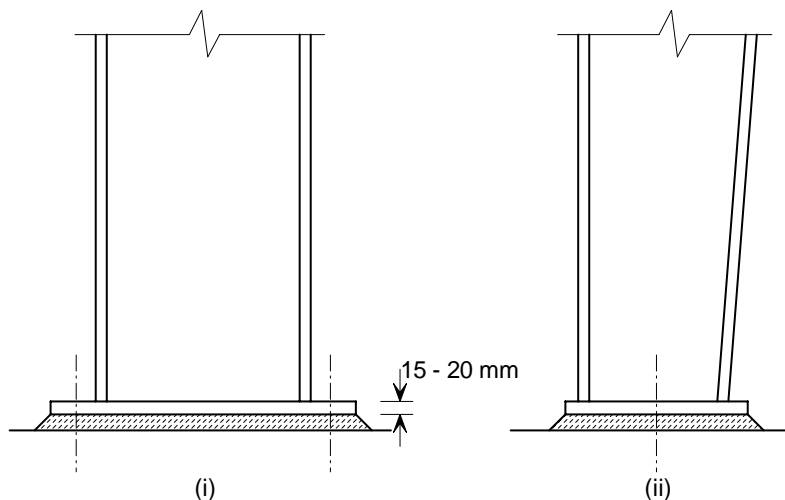
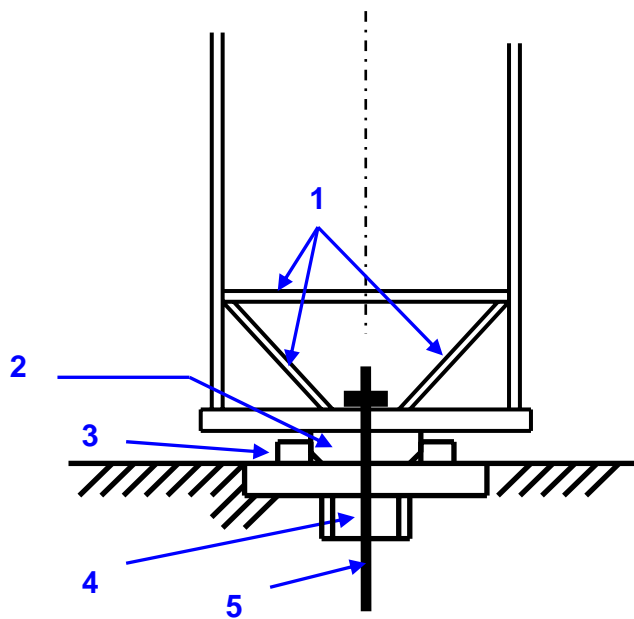


Figure 7.4 Simple "pinned" column bases: (i) constant depth column; (ii) tapered column
Despite their nominal fixity, which enhances safety during construction, these bases have been demonstrated by experience to provide "pinned" performance in practice.



- Key 1: Web stiffeners
2: Bearing shell
3: Bearing shell box
4: Shear nib
5: Anchorage pin

Figure 7.5 *Pinned column base with rocker bearing*

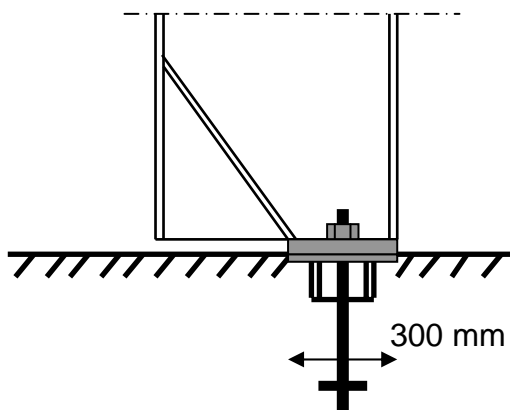


Figure 7.6 *Pinned column base with offset connections to minimise effective frame span and introduce beneficial eccentricity moment to the column*

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