5 Steel elements

5.1 Structural design of steelwork

At present there are two British Standards devoted to the design of structural steel elements:

BS 449 The use of structural steel in building.
BS 5950 Structural use of steelwork in building.

The former employs permissible stress analysis whilst the latter is based upon limit state philosophy. Since it is intended that BS 5950 will eventually replace BS 449, the designs contained in this manual will be based upon BS 5950.

There are to be nine parts to BS 5950:

Part 1 Code of practice for design in simple and continuous construction: hot rolled sections.
Part 2 Specification for materials, fabrication and erection: hot rolled sections.
Part 3 Code of practice for design in composite construction.
Part 4 Code of practice for design of floors with profiled steel sheeting.
Part 5 Code of practice for design of cold formed sections.
Part 6 Code of practice for design in light gauge sheeting, decking and cladding.
Part 7 Specification for materials and workmanship: cold formed sections.
Part 8 Code of practice for design of fire protection for structural steelwork.
Part 9 Code of practice for stressed skin design.

Calculations for the majority of steel members contained in building and allied structures are usually based upon the guidance given in Part 1 of the standard. This manual will therefore be related to that part.

Requirements for the fabrication and erection of structural steelwork are given in Part 2 of the standard. The designer should also be familiar with these, so that he can take into account any which could influence his design.

For information on all aspects of bridge design, reference should be made to BS 5400, 'Steel, concrete and composite bridges'.

The design of a steel structure may be divided into two stages. First the size of the individual members is determined in relation to the induced forces and bending moments. Then all necessary bolted or welded connections are designed so that they are capable of transmitting the forces and bending moments. In this manual we will concentrate on the design of the main structural elements.

Three methods of design are included in BS 5950 Part 1:

Simple design This method applies to structures in which the end connections between members are such that they cannot develop any significant restraint moments. Thus, for the purpose of design, the structure may be considered to be pin-jointed on the basis of the following assumptions:

(a) All beams are simply supported.

(b) All connections are designed to resist only resultant reactions at the appropriate eccentricity.

(c) Columns are subjected to loads applied at the appropriate eccentricity.

(d) Resistance to sway, such as that resulting from lateral wind loads, is provided by either bracing, shear walls or core walls.

Rigid design In this method the structure is considered to be rigidly jointed such that it behaves as a continuous framework. Therefore the connections must be capable of transmitting both forces and bending moments. Portal frames are designed in this manner using either elastic or plastic analysis.

Semi-rigid design This is an empirical method, seldom adopted, which permits partial interaction between beams and columns to be assumed provided that certain stated parameters are satisfied.

The design of steel elements dealt with in this manual will be based upon the principles of simple design.

It is important to appreciate that an economic steel design is not necessarily that which uses the least weight of steel. The most economical solution will be that which produces the lowest overall cost in terms of materials, detailing, fabrication and erection.

5.2 Symbols

The symbols used in BS 5950 and which are relevant to this manual are as follows:

A area
A_g gross sectional area of steel section
A_v shear area (sections)
B breadth of section
b outstand of flange
b_1 stiff bearing length
D depth of section

d	depth of web
E	modulus of elasticity of steel
e	eccentricity
F_c	ultimate applied axial load
F_v	shear force (sections)
I_x	second moment of area about the major axis
I_y	second moment of area about the minor axis
L	length of span
L_E	effective length
M	larger end moment
M_A	maximum moment on the member or portion of the member under consideration
M_b	buckling resistance moment (lateral torsional)
M_{cx}, M_{cy}	moment capacity of section about the major and minor axes in the absence of axial load
M_e	eccentricity moment
M_o	mid-length moment on a simply supported span equal to the unrestrained length
M_u	ultimate moment
M_x	maximum moment occurring between lateral restraints on a beam
$\bar{M}$	equivalent uniform moment
m	equivalent uniform moment factor
n	slenderness correction factor
P_c	compression resistance of column
P_{crip}	ultimate web bearing capacity
P_v	shear capacity of a section
p_b	bending strength
p_c	compressive strength
P_w	buckling resistance of an unstiffened web
p_y	design strength of steel
r_x, r_y	radius of gyration of a member about its major and minor axes
S_x, S_y	plastic modulus about the major and minor axes
T	thickness of a flange or leg
t	thickness of a web or as otherwise defined in a clause
u	buckling parameter of the section
v	slenderness factor for beam
x	torsional index of section
Z_x, Z_y	elastic modulus about the major and minor axes
ß	ratio of smaller to larger end moment
g_f	overall load factor
g_e	load variation factor: function of g_{e1} and g_{e2}
g_m	material strength factor
g	ratio M/M_0, that is the ratio of the larger end moment to the mid-length moment on a simply supported span equal to the unrestrained length
d	deflection
e	constant $(275/p_y)^{1/2}$

λ slenderness, that is the effective length divided by the radius of gyration

λ_{LT} equivalent slenderness

5.3 Definitions

The following definitions which are relevant to this manual have been abstracted from BS 5950 Part 1:

Beam A member predominantly subject to bending.

Buckling resistance Limit of force or moment which a member can withstand without buckling.

Capacity Limit of force or moment which may be applied without causing failure due to yielding or rupture.

Column A vertical member of a structure carrying axial load and possibly moments.

Compact cross-section A cross-section which can develop the plastic moment capacity of the section but in which local buckling prevents rotation at constant moment.

Dead load All loads of constant magnitude and position that act permanently, including self-weight.

Design strength The yield strength of the material multiplied by the appropriate partial factor.

Effective length Length between points of effective restraint of a member multiplied by a factor to take account of the end conditions and loading.

Elastic design Design which assumes no redistribution of moments due to plastic rotation of a section throughout the structure.

Empirical method Simplified method of design justified by experience or testing.

Factored load Specified load multiplied by the relevant partial factor.

H-section A section with one central web and two equal flanges which has an overall depth not greater than 1.2 times the width of the flange.

I-section Section with central web and two equal flanges which has an overall depth greater than 1.2 times the width of the flange.

Imposed load Load on a structure or member other than wind load, produced by the external environment and intended occupancy or use.

Lateral restraint For a beam: restraint which prevents lateral movement of the compression flange. For a column: restraint which prevents lateral movement of the member in a particular plane.

Plastic cross-section A cross-section which can develop a plastic hinge with sufficient rotation capacity to allow redistribution of bending moments within the structure.

Plastic design Design method assuming redistribution of moment in continuous construction.

Semi-compact cross-section A cross-section in which the stress in the extreme fibres should be limited to yield because local buckling would prevent development of the plastic moment capacity in the section.

Serviceability limit states Those limit states which when exceeded can lead to the structure being unfit for its intended use.

Slender cross-section A cross-section in which yield of the extreme fibres cannot be attained because of premature local buckling.

Slenderness The effective length divided by the radius of gyration.

Strength Resistance to failure by yielding or buckling.

Strut A member of a structure carrying predominantly compressive axial load.

Ultimate limit state That state which if exceeded can cause collapse of part or the whole of the structure.

5.4 Steel grades and sections

As mentioned in Chapter 1, steel sections are produced by rolling the steel, whilst hot, into various standard profiles. The quality of the steel that is used must comply with BS 4360 ‘Specification for weldable structural steels’, which designates four basic grades for steel: 40, 43, 50 and 55. (It should be noted that grade 40 steel is not used for structural purposes.) These basic grades are further classified in relation to their ductility, denoted by suffix letters A, B, C and so on. These in turn give grades 43A, 43B, 43C and so on. The examples in this manual will, for simplicity, be based on the use of grade 43A steel.

It is eventually intended to replace the present designations with grade references related to the yield strength of the steel. Thus, for example, grade 43A steel will become grade 275A since it has a yield stress of 275 N/mm^2.

The dimensions and geometric properties of the various hot rolled sections are obtained from the relevant British Standards. Those for universal beam (UB) sections, universal column (UC) sections, rolled steel joist (RSJ) sections and rolled steel channel (RSC) sections are given in BS 4 Part 1. Structural hollow sections and angles are covered by BS 4848 Part 2 and Part 4 respectively. It is eventually intended that BS 4 Part 1 will also become part of BS 4848.

Cold formed steel sections produced from light gauge plate, sheet or strip are also available. Their use is generally confined to special applications and the production of proprietary roof purlins and sheeting rails. Guidance on design using cold formed sections is given in BS 5950 Part 5.

5.5 Design philosophy

The design approach employed in BS 5950 is based on limit state philosophy. The fundamental principles of the philosophy were explained in Chapter 3 in the context of concrete design. In relation to steel structures, some of the ultimate and serviceability limit states (ULSs and SLSs) that may have to be considered are as follows

Ultimate limit states

Strength The individual structural elements should be checked to ensure that they will not yield, rupture or buckle under the influence of the ultimate design loads, forces, moments and so on. This will entail checking beams for the ULSs of bending and shear, and columns for a compressive ULS and when applicable a bending ULS.

Stability The building or structural framework as a whole should be checked to ensure that the applied loads do not induce excessive sway or cause overturning.

Fracture due to fatigue Fatigue failure could occur in a structure that is repeatedly subjected to rapid reversal of stress. Connections are particularly prone to such failure. In the majority of building structures, changes in stress are gradual. However, where dynamic loading could occur, such as from travelling cranes, the risk of fatigue failure should be considered.

Brittle failure Sudden failure due to brittle fracture can occur in steelwork exposed to low temperatures; welded structures are particularly susceptible. Since the steel members in most building frames are protected from the weather, they are not exposed to low temperatures and therefore brittle fracture need not be considered. It is more likely to occur in large welded structures, such as bridges, which are exposed to the extremes of winter temperature. In such circumstances, it is necessary to select steel of adequate notch ductility and to devise details that avoid high stress concentrations.

Serviceability limit states

Deflection Adequate provision must be made to ensure that excessive deflection which could adversely effect any components or finishes supported by the steel members does not occur.

Corrosion and durability Corrosion induced by atmospheric or chemical conditions can adversely affect the durability of a steel structure. The designer must therefore specify a protective treatment suited to the location of the structure. Guidance on the selection of treatments is given in BS 5493 'Code of practice for protective coating of iron and steel structures against corrosion'. Certain classes of grade 50 steel are also available with weather resistant qualities, indicated by the prefix WR, for example WR 50A. Such steel when used in a normal external environment does not need any additional surface protection. An oxide skin forms on the surface of the steel, preventing further corrosion. Provided that the self-coloured appearance is aesthetically acceptable, consideration may be given to its use in situations where exposed steel is permitted, although it should be borne in mind that it is more expensive than ordinary steel.

Fire protection Due consideration should also be given to the provision of adequate protection to satisfy fire regulations. Traditionally fire protection was provided by casing the steelwork in concrete. Nowadays a number of lightweight alternatives are available in the form of dry sheet material, plaster applied to metal lathing, or plaster sprayed directly on to the surface of the steel. Intumescent paints are also marketed which froth when heated to produce a protective insulating layer on the surface of the steel.

Since this manual is concerned with the design of individual structural elements, only the strength ULS and the deflection SLS will be considered further.

5.6 Safety factors

In a similar fashion to concrete and masonry design, partial safety factors are once again applied separately to the loads and material stresses. Initially BS 5950 introduces a third factor, g_p, related to structural performance. The factors given in BS 5950 are as follows:

g_e for load
g_p for structural performance
g_m for material strength.

However, factors g_e and g_p when multiplied together give a single partial safety factor for load of g_f. Hence the three partial safety factors reduce to the usual two of g_f and g_m.

5.7 Loads

The basic loads are referred to in BS 5950 as specified loads rather than characteristic loads. They need to be multiplied by the relevant partial safety factor for load g_f to arrive at the design load.

5.7.1 Specified loads

These are the same as the characteristic loads of dead, imposed and wind previously defined in Chapters 3 and 4 in the context of concrete and masonry design.

5.7.2 Partial safety factors for load

To arrive at the design load, the respective specified loads are multiplied by a partial safety factor g_f in relation to the limit state being considered:

$$\text{Design load} = g_f \times \text{specified load}$$

5.7.3 Ultimate design load

The partial safety factors for the ULS load combinations are given in Table 2 of BS 5950. For the beam and column examples contained in this manual, only the values for the dead and imposed load combination are required, which are 1.4 and 1.6 respectively. Thus the ultimate design load for the dead plus imposed combination would be as follows:

$$\text{Ultimate design load} = g_f \times \text{dead load} + g_f \times \text{imposed load}$$
$$= 1.4 \times \text{dead load} + 1.6 \times \text{imposed load}$$

5.7.4 Serviceability design load

For the purpose of checking the deflection SLS, the partial safety factor g_f may be taken as unity. Furthermore, in accordance with BS 5950, the deflection of a beam need only be checked for the effect of imposed loading. Hence the serviceability design load for checking the deflection of a steel beam is simply the specified imposed load. This differs from the design of timber and concrete beams, for which the dead plus imposed load is used to check deflection. However, it is not unreasonable since we are only interested in controlling the deflection of steel beams to avoid damage to finishes, and the dead load deflection will already have taken place before these are applied. If for reasons of appearance it is considered necessary to counteract all or part of the dead load deflection, the beam could be pre-cambered.

5.8 Material properties

The ultimate design strength p_y for the most common types of structural steel are given in BS 5950 Table 6, from which those for grade 43 steel are shown here in Table 5.1. They incorporate the material partial safety factor g_m in the specified values. Therefore the strength may be obtained directly from the table without further modification. For beam and column sections the material thickness referred to in the table should be taken as the flange thickness.

Table 5.1 Design strength p_y of grade 43 steel

Thickness less than or equal to (mm)	p_y for rolled sections, plates and hollow sections (N/mm^2)
16	275
40	265
63	255
100	245

The modulus of elasticity E, for deflection purposes, may be taken as 205 kN/mm^2 for all grades of steel.

5.9 Section properties

Dimensions and geometric properties for the hot rolled steel sections commonly available for use as beams and columns are tabulated in BS 4 Part 1. Similar tables expanded to include a number of useful design constants are also published by the Steel Construction Institute. These are contained in their *Steelwork Design Guide to BS 5950: Part 1*, Volume 1, *Section Properties, Member Capacities.* Tables 5.2 and 5.3 given here are typical examples from that publication, reproduced by kind permission of the director of the Steel Construction Institute. Complete copies of the guide can be obtained from the Institute at Silwood Park, Ascot, Berkshire, SL5 7QN.

Table 5.2 relates to universal beam (UB) sections, as illustrated in Figure 5.1, and Table 5.3 to universal column (UC) sections, as illustrated in Figure 5.2. The use of these tables in relation to the design of beams and columns will be explained in the appropriate sections of this chapter.

Whilst the UB sections are primarily intended for use as beams, they can if desired be used as columns; this is often the case in portal frame construction. Similarly the UC sections are intended for use as columns but can also be used as beams. However, because they have a stocky cross-section they do not lend themselves as readily to such an alternative use.

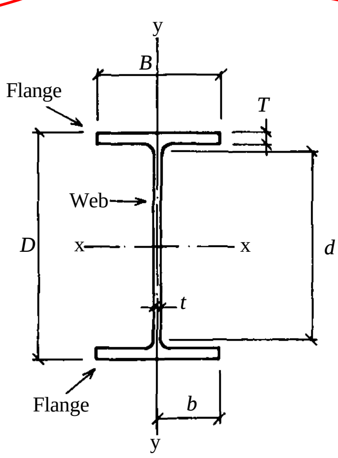


Figure 5.1 Universal beam cross-section

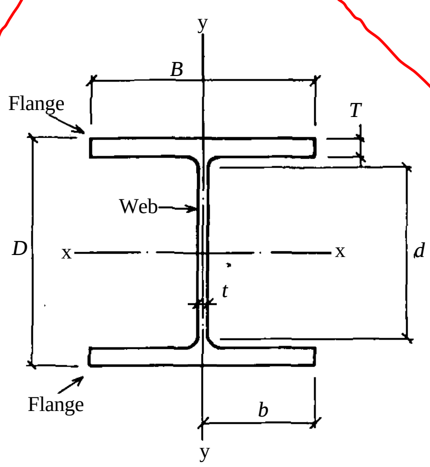


Figure 5.2 *Universal column cross-section*

5.10 Beams

The main structural design requirements for which steel beams should be examined as as follows:

(a) Bending ULS

Table 5.2 Universal beams (abstracted from the *Steelwork Design Guide to BS 59.50: Part 1*, published by the Steel Construction Institute)

(a) Dimensions

Designation		Depth of section	Width of section	Thickness		Root radius	Depth between fillets	Ratios for local buckling		Dimensions for detailing			Surface area	
Serial size	Mass per metre	D	B	Web t	Flange T	r	d	Flange b/T	Web d/t	End clearance C	Notch N	Notch n	Per metre	per tonne
(mm)	(kg)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)			(mm)	(mm)	(mm)	(m^2)	(m^2)
914 × 419	388	920.5	420.5	21.5	36.6	24.1	799.1	5.74	37.2	13	210	62	3.44	8.86
	343	911.4	418.5	19.4	32.0	24.1	799.1	6.54	41.2	12	210	58	3.42	9.96
914 × 305	289	926.6	307.8	19.6	32.0	19.1	824.5	4.81	42.1	12	156	52	3.01	10.4
	253	918.5	305.5	17.3	27.9	19.1	824.5	5.47	47.7	11	156	48	2.99	11.8
	224	910.3	304.1	15.9	23.9	19.1	824.5	6.36	51.9	10	156	44	2.97	13.3
	201	903.0	303.4	15.2	20.2	19.1	824.5	7.51	54.2	10	156	40	2.96	14.7
838 × 292	226	850.9	293.8	16.1	26.8	17.8	761.7	5.48	47.3	10	150	46	2.81	12.5
	194	840.7	292.4	14.7	21.7	17.8	761.7	6.74	51.8	9	150	40	2.79	14.4
	176	834.9	291.6	14.0	18.8	17.8	761.7	7.76	54.5	9	150	38	2.78	15.8
762 × 267	197	769.6	268.0	15.6	25.4	16.5	685.8	5.28	44.0	10	138	42	2.55	13.0
	173	762.0	266.7	14.3	21.6	16.5	685.8	6.17	48.0	9	138	40	2.53	14.6
	147	753.9	265.3	12.9	17.5	16.5	685.8	7.58	53.2	8	138	36	2.51	17.1
686 × 254	170	692.9	255.8	14.5	23.7	15.2	615.1	5.40	42.4	9	132	40	2.35	13.8
	152	687.6	254.5	13.2	21.0	15.2	615.1	6.06	46.6	9	132	38	2.34	15.4
	140	683.5	253.7	12.4	19.0	15.2	615.1	6.68	49.6	8	132	36	2.33	16.6
	125	677.9	253.0	11.7	16.2	15.2	615.1	7.81	52.6	8	132	32	2.32	18.5
610 × 305	238	633.0	311.5	18.6	31.4	16.5	537.2	4.96	28.9	11	158	48	2.45	10.3
	179	617.5	307.0	14.1	23.6	16.5	537.2	6.50	38.1	9	158	42	2.41	13.4
	149	609.6	304.8	11.9	19.7	16.5	537.2	7.74	45.1	8	158	38	2.39	16.0
610 × 229	140	617.0	230.1	13.1	22.1	12.7	547.3	5.21	41.8	9	120	36	2.11	15.0
	125	611.9	229.0	11.9	19.6	12.7	547.3	5.84	46.0	8	120	34	2.09	16.8
	113	607.3	228.2	11.2	17.3	12.7	547.3	6.60	48.9	8	120	32	2.08	18.4
	101	602.2	227.6	10.6	14.8	12.7	547.3	7.69	51.6	7	120	28	2.07	20.5
533 × 210	122	544.6	211.9	12.8	21.3	12.7	476.5	4.97	37.2	8	110	36	1.89	15.5
	109	539.5	210.7	11.6	18.8	12.7	476.5	5.60	41.1	8	110	32	1.88	17.2
	101	536.7	210.1	10.9	17.4	12.7	476.5	6.04	43.7	7	110	32	1.87	18.5
	92	533.1	209.3	10.2	15.6	12.7	476.5	6.71	46.7	7	110	30	1.86	20.2
	82	528.3	208.7	9.6	13.2	12.7	476.5	7.91	49.6	7	110	26	1.85	22.6
457 × 191	98	467.4	192.8	11.4	19.6	10.2	407.9	4.92	35.8	8	102	30	1.67	17.0
	89	463.6	192.0	10.6	17.7	10.2	407.9	5.42	38.5	7	102	28	166	18.6
	82	460.2	191.3	9.9	16.0	10.2	407.9	5.98	41.2	7	102	28	1.65	20.1
	74	457.2	190.5	9.1	14.5	10.2	407.9	6.57	44.8	7	102	26	1.64	22.2
	67	453.6	189.9	8.5	12.7	10.2	407.9	7.48	48.0	6	102	24	1.63	24.4
457 × 152	82	465.1	153.5	10.7	18.9	10.2	407.0	4.06	38.0	7	82	30	1.51	18.4
	74	461.3	152.7	9.9	17.0	10.2	407.0	4.49	41.1	7	82	28	1.50	20.2
	67	457.3	151.9	9.1	15.0	10.2	407.0	5.06	44.7	7	82	26	1.49	22.2
	60	454.7	152.9	8.0	13.3	10.2	407.0	5.75	51.0	6	84	24	1.49	24.8
	52	449.8	152.4	7.6	10.9	10.2	407.0	6.99	53.6	6	84	22	1.48	28.4

Table 5.2 Universal beams *continued* (abstracted from the *Steelwork Design Guide to BS 5950: Part 1*, published by the Steel Construction Institute)

(b) Properties

Designation		Second moment of area		Radius of gyration		Elastic modulus		Plastic modulus		Buckling parameter	Torsional index	Warping constant	Torsional constant	Area of section
Serial size	Mass per metre	Axis x–x	Axis y–y	Axis x–x	Axis y–y	Axis x–x	Axis y–y	Axis x–x	Axis y–y	u	x	H	J	A
(mm)	(kg)	(cm^4)	(cm^4)	(cm)	(cm)	(cm^3)	(cm^3)	(cm^3)	(cm^3)			(dm^6)	(cm^4)	(cm^2)
914 × 419	388	719 000	45 400	38.1	9.58	15 600	2160	17 700	3340	0.884	26.7	88.7	1730	494
	343	625 000	39 200	37.8	9.46	13 700	1870	15 500	2890	0.883	30.1	75.7	1190	437
914 × 305	289	505 000	15 600	37.0	6.51	10 900	1010	12 600	1600	0.867	31.9	31.2	929	369
	253	437 000	13 300	36.8	6.42	9 510	872	10 900	1370	0.866	36.2	26.4	627	323
	224	376 000	11 200	36.3	6.27	8 260	738	9 520	1160	0.861	41.3	22.0	421	285
	201	326 000	9 430	35.6	6.06	7 210	621	8 360	983	0.853	46.8	18.4	293	256
838 × 292	226	340 000	11 400	34.3	6.27	7 990	773	9 160	1210	0.87	35.0	19.3	514	289
	194	279 000	9 070	33.6	6.06	6 650	620	7 650	974	0.862	41.6	15.2	307	247
	176	246 000	7 790	33.1	5.90	5 890	534	6 810	842	0.856	46.5	13.0	222	224
762 × 267	197	240 000	8 170	30.9	5.71	6 230	610	7 170	959	0.869	33.2	11.3	405	251
	173	205 000	6 850	30.5	5.57	5 390	513	6 200	807	0.864	38.1	9.38	267	220
	147	169 000	5 470	30.0	5.39	4 480	412	5 170	649	0.857	45.1	7.41	161	188
686 × 254	170	170 000	6 620	28.0	5.53	4 910	518	5 620	810	0.872	31.8	7.41	307	217
	152	150 000	5 780	27.8	5.46	4 370	454	5 000	710	0.871	35.5	6.42	219	194
	140	136 000	5 180	27.6	5.38	3 990	408	4 560	638	0.868	38.7	5.72	169	179
	125	118 000	4 380	27.2	5.24	3 480	346	4 000	542	0.862	43.9	4.79	116	160
610 × 305	238	208 000	15 800	26.1	7.22	6 560	1020	7 460	1570	0.886	21.1	14.3	788	304
	179	152 000	11 400	25.8	7.08	4 910	743	5 520	1140	0.886	27.5	10.1	341	228
	149	125 000	9 300	25.6	6.99	4 090	610	4 570	937	0.886	32.5	8.09	200	190
610 × 229	140	112 000	4 510	25.0	5.03	3 630	392	4 150	612	0.875	30.5	3.99	217	178
	125	98 600	3 930	24.9	4.96	3 220	344	3 680	536	0.873	34.0	3.45	155	160
	113	87 400	3 440	24.6	4.88	2 880	301	3 290	470	0.87	37.9	2.99	112	144
	101	75 700	2 910	24.2	4.75	2 510	256	2 880	400	0.863	43.0	2.51	77.2	129
533 × 210	122	76 200	3 390	22.1	4.67	2 800	320	3 200	501	0.876	27.6	2.32	180	156
	109	66 700	2 940	21.9	4.60	2 470	279	2 820	435	0.875	30.9	1.99	126	139
	101	61 700	2 690	21.8	4.56	2 300	257	2 620	400	0.874	33.1	1.82	102	129
	92	55 400	2 390	21.7	4.51	2 080	229	2 370	356	0.872	36.4	1.60	76.2	118
	82	47 500	2 010	21.3	4.38	1 800	192	2 060	300	0.865	41.6	1.33	51.3	104
457 × 191	98	45 700	2 340	19.1	4.33	1 960	243	2 230	378	0.88	25.8	1.17	121	125
	89	41 000	2 090	19.0	4.28	1 770	217	2 010	338	0.879	28.3	1.04	90.5	114
	82	37 100	1 870	18.8	4.23	1 610	196	1 830	304	0.877	30.9	0.923	69.2	105
	74	33 400	1 670	18.7	4.19	1 460	175	1 660	272	0.876	33.9	0.819	52.0	95.0
	67	29 400	1 450	18.5	4.12	1 300	153	1 470	237	0.873	37.9	0.706	37.1	85.4
457 × 152	82	36 200	1 140	18.6	3.31	1 560	149	1 800	235	0.872	27.3	0.569	89.3	104
	74	32 400	1 010	18.5	3.26	1 410	133	1 620	209	0.87	30.0	0.499	66.6	95.0
	67	28 600	878	18.3	3.21	1 250	116	1 440	182	0.867	33.6	0.429	47.5	85.4
	60	25 500	794	18.3	3.23	1 120	104	1 280	163	0.869	37.5	0.387	33.6	75.9
	52	21 300	645	17.9	3.11	949	84.6	1 090	133	0.859	43.9	0.311	21.3	66.5

Table 5.3 Universal columns (abstracted from the *Steelwork Design Guide to BS 5950: Part 1*, published by the Steel Construction Institute)

(a) Dimensions

Designation		Depth Of section	Width Of section	Thickness		Root radius	Depth between fillets	Ratios for local buckling		Dimensions for detailing			Surface area	
Serial size	Mass per metre	D	B	Web t	Flange T	r	d	Flange b/T	Web d/t	End clearance C	Notch N	Notch n	Per metre	per tonne
(mm)	(kg)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)			(mm)	(mm)	(mm)	(m^2)	(m^2)
356 × 406	634	474.7	424.1	47.6	77.0	15.2	290.2	2.75	6.10	26	200	94	2.52	3.98
	551	455.7	418.5	42.0	67.5	15.2	290.2	3.10	6.91	23	200	84	2.48	4.49
	467	436.6	412.4	35.9	58.0	15.2	290.2	3.56	8.08	20	200	74	2.42	5.19
	393	419.1	407.0	30.6	49.2	15.2	290.2	4.14	9.48	17	200	66	2.38	6.05
	340	406.4	403.0	26.5	42.9	15.2	290.2	4.70	11.0	15	200	60	2.35	6.90
	287	393.7	399.0	22.6	36.5	15.2	290.2	5.47	12.8	13	200	52	2.31	8.06
	235	381.0	395.0	18.5	30.2	15.2	290.2	6.54	15.7	11	200	46	2.28	9.70
COLCORE	477	427.0	424.4	48.0	53.2	15.2	290.2	3.99	6.05	26	200	70	2.43	5.09
356 × 368	202	374.7	374.4	16.8	27.0	15.2	290.2	6.93	17.3	10	190	44	2.19	10.8
	177	368.3	372.1	14.5	23.8	15.2	290.2	7.82	20.0	9	190	40	2.17	12.3
	153	362.0	370.2	12.6	20.7	15.2	290.2	8.94	23.0	8	190	36	2.15	14.1
	129	355.6	368.3	10.7	17.5	15.2	290.2	10.5	27.1	7	190	34	2.14	16.6
305 × 305	283	365.3	321.8	26.9	44.1	15.2	246.6	3.65	9.17	15	158	60	1.94	6.85
	240	352.6	317.9	23.0	37.7	15.2	246.6	4.22	10.7	14	158	54	1.90	7.93
	198	339.9	314.1	19.2	31.4	15.2	246.6	5.00	12.8	12	158	48	1.87	9.45
	158	327.2	310.6	15.7	25.0	15.2	246.6	6.21	15.7	10	158	42	1.84	11.6
	137	320.5	308.7	13.8	21.7	15.2	246.6	7.11	17.9	9	158	38	1.82	13.3
	118	314.5	306.8	11.9	18.7	15.2	246.6	8.20	20.7	8	158	34	1.81	5.3
	97	307.8	304.8	9.9	15.4	15.2	246.6	9.90	24.9	7	158	32	1.79	18.4
254 × 254	167	289.1	264.5	19.2	31.7	12.7	200.3	4.17	10.4	12	134	46	1.58	9.44
	132	276.4	261.0	15.6	25.3	12.7	200.3	5.16	12.8	10	134	40	1.54	11.7
	107	266.7	258.3	13.0	20.5	12.7	200.3	6.30	15.4	9	134	34	1.52	14.2
	89	260.4	255.9	10.5	17.3	12.7	200.3	7.40	19.1	7	134	32	1.50	16.9
	13	254.0	254.0	8.6	14.2	12.7	200.3	8.94	23.3	6	134	28	1.49	20.3
203 × 203	86	222.3	208.8	13.0	20.5	10.2	160.9	5.09	12.4	9	108	32	1.24	14.4
	71	215.9	206.2	10.3	17.3	10.2	160.9	5.96	15.6	7	108	28	1.22	17.2
	60	209.6	205.2	9.3	14.2	10.2	160.9	7.23	17.3	7	108	26	1.20	20.1
	52	206.2	203.9	8.0	12.5	10.2	160.9	8.16	20.1	6	108	24	1.19	23.0
	46	203.2	203.2	7.3	11.0	10.2	160.9	9.24	22.0	6	108	22	1.19	25.8
152 × 152	37	161.8	154.4	8.1	11.5	7.6	123.5	6.71	15.2	6	84	20	0.912	24.6
	30	157.5	152.9	6.6	9.4	7.6	123.5	8.13	18.7	5	84	18	0.9	30.0
	23	152.4	152.4	6.1	6.8	7.6	123.5	11.2	20.2	5	84	16	0.889	38.7

Table 5.3 Universal columns *continued* (abstracted from the *Steelwork Design Guide to BS 5950: Part 1*, published by the Steel Construction Institute)

(b) Properties

Designation		Second moment of area		Radius of gyration		Elastic modulus		Plastic modulus		Buckling parameter	Torsional index	Warping constant	Torsional constant	Area of section
Serial size	Mass per metre	Axis *x–x*	Axis *y–y*	Axis *x–x*	Axis *y–y*	Axis *x–x*	Axis *y–y*	Axis *x–x*	Axis *y–y*	*u*	*x*	*H*	*J*	*A*
(mm)	(kg)	(cm^4)	(cm^4)	(cm)	(cm)	(cm^3)	(cm^3)	(cm^3)	(cm^3)			(dm^6)	(cm^4)	(cm^2)
356 × 406	634	275 000	98 200	18.5	11.0	11 600	4630	14 200	7110	0.843	5.46	38.8	13 700	808
	551	227 000	82 700	18.0	10.9	9 960	3950	12 100	6060	0.841	6.05	31.1	9 240	702
	467	183 000	67 900	17.5	10.7	8 390	3290	10 000	5040	0.839	6.86	24.3	5 820	595
	393	147 000	55 400	17.1	10.5	7 000	2720	8 230	4160	0.837	7.86	19.0	3 550	501
	340	122 000	46 800	16.8	10.4	6 030	2320	6 990	3540	0.836	8.85	15.5	2 340	433
	287	100 000	38 700	16.5	10.3	5 080	1940	5 820	2950	0.835	10.2	12.3	1 440	366
	235	79 100	31 000	16.2	10.2	4 150	1570	4 690	2380	0.834	12.1	9.54	812	300
COLCORE	477	172 000	68 100	16.8	10.6	8 080	3210	9 700	4980	0.815	6.91	23.8	5 700	607
356 × 368	202	66 300	23 600	16.0	9.57	3 540	1260	3 980	1920	0.844	13.3	7.14	560	258
	177	57 200	20 500	15.9	9.52	3 100	1100	3 460	1670	0.844	15.0	6.07	383	226
	153	48 500	17 500	15.8	9.46	2 680	944	2 960	1430	0.844	17.0	5.09	251	195
	129	40 200	14 600	15.6	9.39	2 260	790	2 480	1200	0.843	19.9	4.16	153	165
305 × 305	283	78 800	24 500	14.8	8.25	4 310	1530	5 100	2340	0.855	7.65	6.33	2 030	360
	240	64 200	20 200	14.5	8.14	3 640	1270	4 250	1950	0.854	8.73	5.01	1 270	306
	198	50 800	16 200	14.2	8.02	2 990	1030	3 440	1580	0.854	10.2	3.86	734	252
	158	38 700	12 500	13.9	7.89	2 370	806	2 680	1230	0.852	12.5	2.86	379	201
	137	32 800	10 700	13.7	7.82	2 050	691	2 300	1050	0.851	14.1	2.38	250	175
	118	27 600	9 010	13.6	7.75	1 760	587	1 950	892	0.851	16.2	1.97	160	150
	97	22 200	7 270	13.4	7.68	1 440	477	1 590	723	0.850	19.3	1.55	91.1	123
254 × 254	167	29 900	9 800	11.9	6.79	2 070	741	2 420	1130	0.852	8.49	1.62	625	212
	132	22 600	7 520	11.6	6.67	1 630	576	1 870	879	0.850	10.3	1.18	322	169
	107	17 500	5 900	11.3	6.57	1 310	457	1 490	695	0.848	12.4	0.894	173	137
	89	14 300	4 850	11.2	6.52	1 100	379	1 230	575	0.849	14.4	0.716	104	114
	73	11 400	3 870	11.1	6.46	894	305	989	462	0.849	17.3	0.557	57.3	92.9
203 × 203	86	9 460	3 120	9.27	5.32	851	299	979	456	0.85	10.2	0.317	138	110
	71	7 650	2 540	9.16	5.28	708	246	802	374	0.852	11.9	0.25	81.5	91.1
	60	6 090	2 040	8.96	5.19	581	199	652	303	0.847	14.1	0.195	46.6	75.8
	52	5 260	1 770	8.90	5.16	510	174	568	264	0.848	15.8	0.166	32.0	66.4
	46	4 560	1 540	8.81	5.11	449	151	497	230	0.846	17.7	0.142	22.2	58.8
152 × 152	37	2 220	709	6.84	3.87	274	91.8	310	140	0.848	13.3	0.04	19.5	47.4
	30	1 740	558	6.75	3.82	221	73.1	247	111	0.848	16.0	0.0306	10.5	38.2
	23	1 260	403	6.51	3.68	166	52.9	184	80.9	0.837	20.4	0.0214	4.87	29.8

(b) Shear ULS
(c) Deflection SLS.

Two other ultimate limit state factors that should be given consideration are:

(d) Web buckling resistance
(e) Web bearing resistance.

However, these are not usually critical under normal loading conditions, and in any case may be catered for by the inclusion of suitably designed web stiffeners.

Let us consider how each of these requirements influences the design of beams.

5.10.1 Bending ULS

When a simply supported beam bends, the extreme fibres above the neutral axis are placed in compression. If the beam is a steel beam this means that the top flange of the section is in compression and correspondingly the bottom flange is in tension. Owing to the combined effect of the resultant compressive loading and the vertical loading, the top flange could tend to deform sideways and twist about its longitudinal axis as illustrated in Figure 5.3. This is termed lateral torsional buckling, and could lead to premature failure of the beam before it reaches its vertical moment capacity.

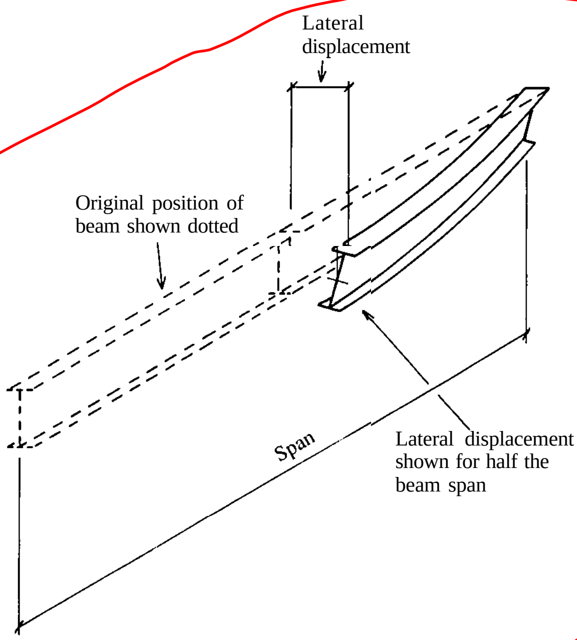


Figure 5.3 *Lateral torsional buckling*

Lateral torsional buckling can be avoided by fully restraining the compression flange along its entire length (Figure 5.4). Alternatively, transverse restraint members can be introduced along the span of the beam (Figure 5.5). These must be at sufficient intervals to prevent lateral torsional buckling occurring between the points of restraint. If neither of these measures are adopted then the beam must be considered as laterally unrestrained and its resistance to lateral torsional buckling should be checked. The requirements that must be fulfilled by both lateral and torsional restraints are described in BS 5950.

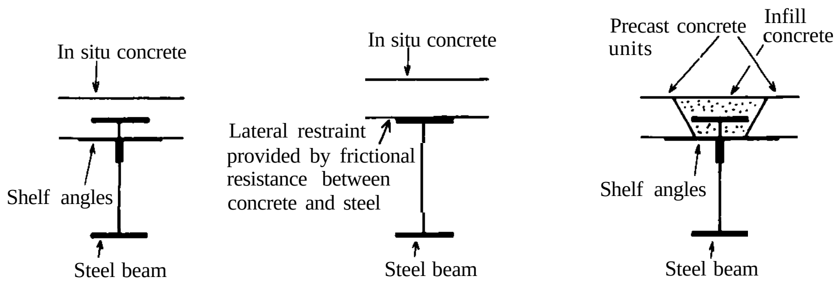


Figure 5.4 *Cross-sections through fully laterally restrained beams*

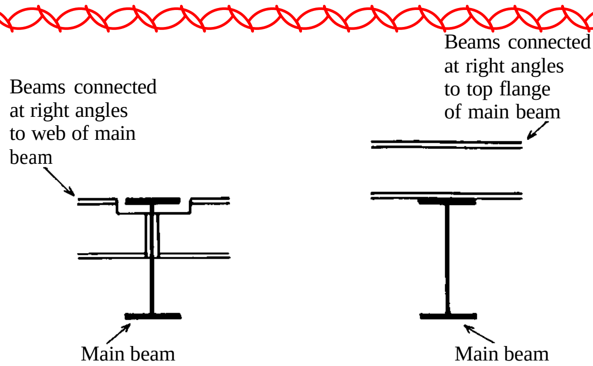


Figure 5.5 *Cross-sections through beams laterally restrained at intervals along their length*

It can be seen from the foregoing that it is necessary to investigate the bending ULS of steel beams in one of two ways: laterally restrained and laterally unrestrained. These are now discussed in turn.

5.10.2 Bending ULS of laterally restrained beams

It has already been shown in Chapter 1 that, in relation to the theory of bending, the elastic moment of resistance (MR) of a steel beam is given by

$$\mathrm{MR} = fZ$$

where f is the permissible bending stress value for the steel and Z is the elastic modulus of the section. This assumes that the elastic stress distribution over the depth of the section will be a maximum at the extreme fibres and zero at the neutral axis (NA), as shown in Figure 5.6.

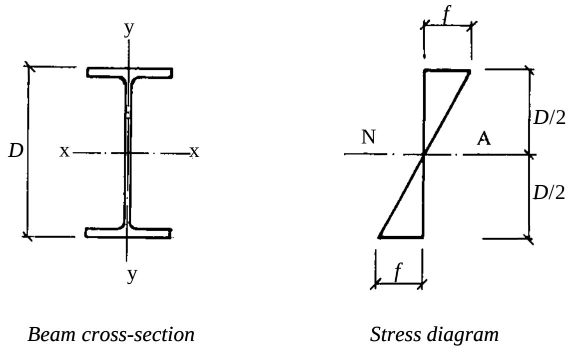


Figure 5.6 *Elastic stress distribution*

To ensure the adequacy of a particular steel beam, its internal moment of resistance must be equal to or greater than the applied bending moment:

$$MR \geq BM \text{ (calculated bending moment)}$$

This was the method employed in previous Codes of Practice for steel design based upon permissible stress analysis.

In limit state design, advantage is taken of the ability of many steel sections to carry greater loads up to a limit where the section is stressed to yield throughout its depth, as shown in Figure 5.7. The section in such a case is said to have become fully plastic. The moment capacity of such a beam about its major x–x axis would be given by

$$M_{cx} = p_y S_x$$

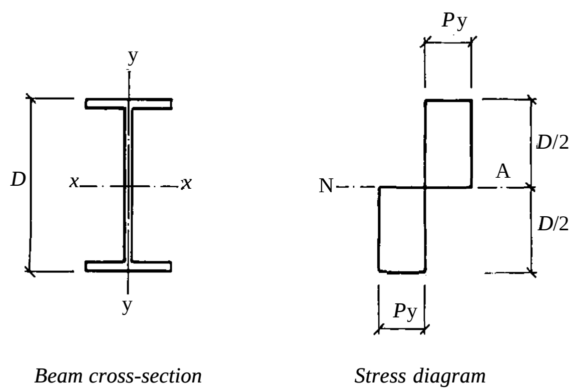


Figure 5.7 *Plastic stress distribution*

where p_y is the design strength of the steel, given in Table 5.1, and S_x is the plastic modulus of the section about the major axis, obtained from section tables. In order that plasticity at working load does not occur before the ultimate load is reached, BS 5950 places a limit on the moment capacity of 1.2 $p_y Z$. Thus

$$M_{cx} = p_y S_x \leq 1.2\, p_y Z_x$$

The suitability of a particular steel beam would be checked by ensuring that the moment capacity of the section is equal to or greater than the applied ultimate moment M_u:

$$M_{cx} \geq M_u$$

The web and flanges of steel sections are comparatively slender in relation to their depth and breadth. Consequently the compressive force induced in a beam by bending could cause local buckling of the web or flange before the full plastic stress is developed. This must not be confused with the previously mentioned lateral torsional buckling, which is a different mode of failure and will be dealt with in the next section. Nor should it be confused with the web buckling ULS discussed in Section 5.10.6.

Local buckling may be avoided by reducing the stress capacity of the section, and hence its moment capacity, relative to its susceptibility to local buckling failure. In this respect steel sections are classified by BS 5950 in relation to the b/T of the flange and the d/t of the web, where b, d, T and t are as previously indicated in Figures 5.1 and 5.2. There are four classes of section:

Class 1 Plastic
Class 2 Compact
Class 3 Semi-compact
Class 4 Slender.

The limiting width to thickness ratios for classes 1, 2 and 3 are given in BS 5950 Table 7, for both beams and columns. Those for rolled beams are listed here in Table 5.4.

Table 5.4 Beam cross-section classification

Limiting proportions	Class of section: Plastic	Compact	Semi-compact
b/T	8.5 e	9.5 e	15 e
d/t	79 e	98 e	120 e

The constant $e = (275/p_y)^{1/2}$. Hence for grade 43 steel:
When $T \leq 16$ mm, $e = (275/275)^{1/2} = 1$
When $T > 16$ mm, $e = (275/265)^{1/2} = 1.02$

Slender sections are those whose thickness ratios exceed the limits for semi-compact sections. Their design strength p_y has to be reduced using a stress reduction factor for slender elements, obtained from Table 8 of BS 5950.

The stress distribution and moment capacity for each class of section is shown in Figure 5.8.

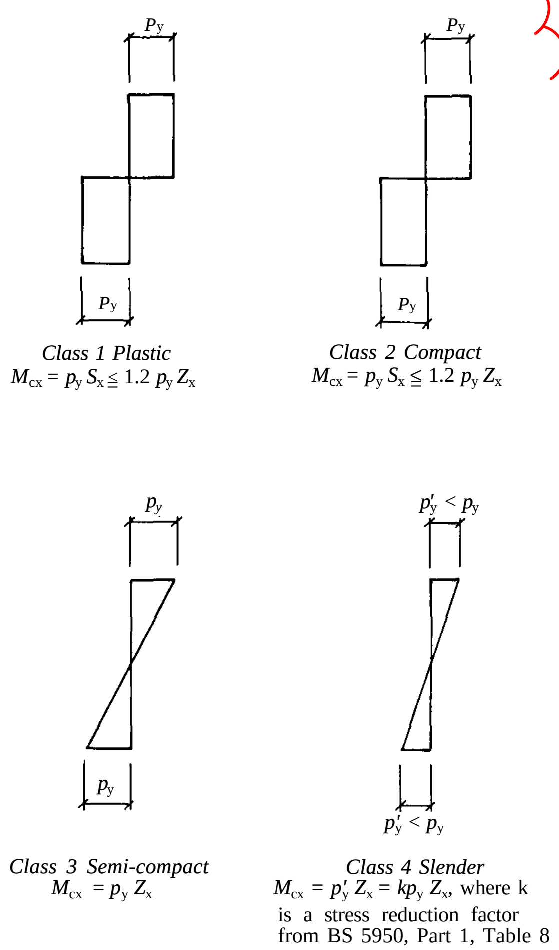


Figure 5.8 *Stress distribution diagrams and moment capacities for section classes*

The examples contained in this manual are based upon the use of grade 43 steel sections. All the UB sections formed from grade 43 steel satisfy either the plastic or the compact classification parameters, and hence the stress reduction factor for slender elements does not apply. Furthermore, their plastic modulus S_x never exceeds 1.2 times their elastic modulus Z_x. Therefore the moment capacity of grade 43 beams will be given by the expression

$$M_{cx} = p_y S_x$$

By rearranging this expression, the plastic modulus needed for a grade 43 UB section to resist a particular ultimate moment may be determined:

$$S_x \text{ required} = \frac{M_u}{p_y}$$

Example 5.1

Steel floor beams arranged as shown in Figure 5.9 support a 150 mm thick reinforced concrete slab which fully restrains the beams laterally. If the floor has to support a specified imposed load of 5 kN/m^2 and reinforced concrete weighs 2400 kg/m^3, determine the size of grade 43 UBs required.

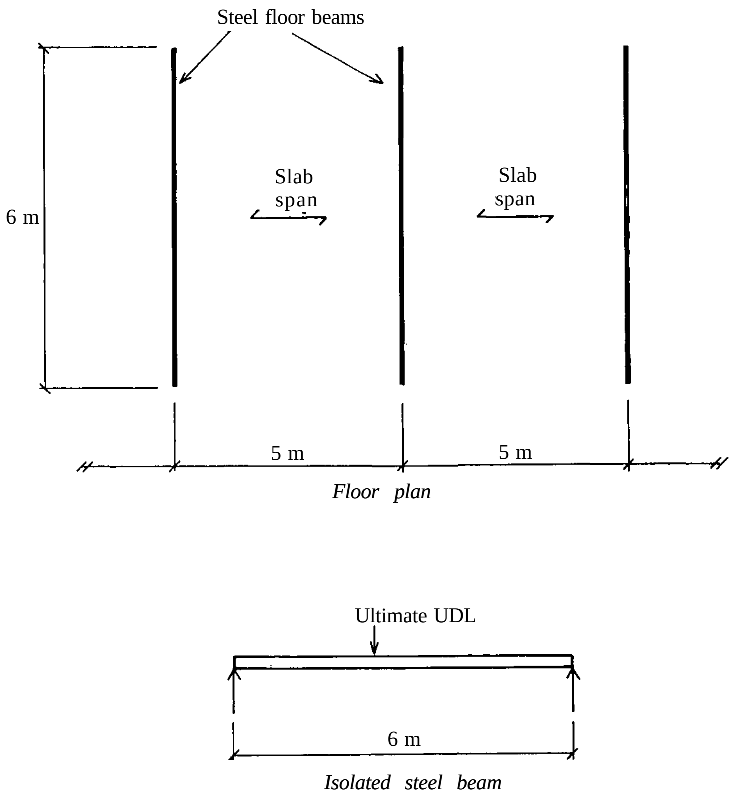


Figure 5.9 *Floor beam arrangement*

Before proceeding to the design of the actual beams it is first necessary to calculate the ultimate design load on an individual beam. This basically follows the procedure explained in Chapter 1, except that partial safety factors for load γ_f need to be applied since we are using limit state design.

Specified dead load 150 mm slab = 0.15 × 2400/100 = 3.6 kN/m^2

Specified dead load UDL = (3.6 × 6 × 5) + SW = 108 + say 4 = 112 kN

Specified imposed load = 5 kN/m^2

Specified imposed load UDL = 5 × 6 × 5 = 150 kN

Total ULS design load = γ_f × specified dead load + γ_f × specified imposed load
= 1.4 × 112 + 1.6 × 150 = 156.8 + 240 = 396.8 kN

Ultimate bending moment $M_u = \frac{WL}{8} = \frac{396.8 \times 6}{8}$
$= 297.6$ kN m $= 297.6 \times 10^6$ N mm

The ultimate design strength p_y for grade 43 steel sections, from Table 5.1, is 275 N/mm^2 provided that the flange thickness does not exceed 16 mm. If the flange thickness was greater than 16 mm, p_y would reduce to 265 N/mm^2. Hence the plastic modulus is

$$S_x \text{ required} = \frac{M_u}{p_y} = \frac{297.6 \times 10^6}{275} = 1\,082\,182 \text{ mm}^3 = 1082 \text{ cm}^3$$

It should be appreciated that the plastic modulus property is always tabulated in cm^3 units.

By reference to Table 5.2, the lightest UB section with a plastic modulus greater than that required is a 457 × 152 × 60 kg/m UB with an S_x of 1280 cm^3. It should be noted that the flange thickness of the selected section is 13.3 mm; this is less than 16 mm, and it was therefore correct to adopt a p_y of 275 N/mm^2 in the design. It should also be noted that the self-weight of the section is less than that assumed and therefore no adjustment to the design is necessary; that is,

$$\text{SW} = \frac{60}{100} \times 6 = 3.6 \text{ kN} < 4 \text{ kN assumed}$$

This section would be adopted provided that it could also satisfy the shear and deflection requirements which will be discussed later.

The design approach employed in Example 5.1 only applies to beams which are fully restrained laterally and are subject to low shear loads. When plastic and compact beam sections are subject to high shear loads their moment capacity reduces because of the interaction between shear and bending. Modified expressions are given in BS 5950 for the moment capacity of beams in such circumstances. However, except for heavily loaded short span beams, this is not usually a problem and it will therefore not be given any further consideration here.

5.10.3 Bending ULS of laterally unrestrained beams

Laterally unrestrained beams are susceptible to lateral torsional buckling failure, and must therefore be designed for a lower moment capacity known as the buckling resistance moment M_b. It is perhaps worth reiterating that torsional buckling is not the same as local buckling, which also needs to be taken into account by reference to the section classification of plastic, compact, semi-compact or slender.

For rolled universal sections or joists BS 5950 offers two alternative approaches – rigorous or conservative – for the assessment of a member's lateral torsional buckling resistance. The rigorous approach may be applied to any form of section acting as a beam, whereas the conservative approach applies only to UB, UC and RSJ sections. Let us therefore consider the implications of each of these approaches with respect to the design of rolled universal sections.

Laterally unrestrained beams, rigorous approach

Unlike laterally restrained beams, it is the section's buckling resistance moment M_b that is usually the criterion rather than its moment capacity M_c. This is given by the following expression:

$$M_b = p_b S_x$$

where p_b is the bending strength and S_x is the plastic modulus of the section about the major axis, obtained from section tables.

The bending strength of laterally unrestrained rolled sections is obtained from BS 5950 Table 11, reproduced here as Table 5.5. It depends on the steel design strength p_y and the equivalent slenderness l_{LT}, which is derived from the following expression:

$$l_{LT} = nuvl$$

where

- n slenderness correction factor from BS 5950
- u buckling parameter of the section, found from section tables or conservatively taken as 0.9
- v slenderness factor from BS 5950
- l minor axis slenderness: $l = L_E/r_y$
- L_E effective unrestrained length of the beam
- r_y radius of gyration of the section about its minor axis, from section tables

The effective length L_E should be obtained in accordance with one of the following conditions:

Condition (a). For beams with lateral restraints at the ends only, the value of L_E should be obtained from BS 5950 Table 9, reproduced here as Table 5.6, taking L as the span of the beam. Where the restraint conditions at each end of the beam differ, the mean value of L_E should be taken.

Condition (b). For beams with effective lateral restraints at intervals along their length, the value of L_E should be taken as 1.0 L for normal loading conditions or 1.2 L for destabilizing conditions, taking L as the distance between restraints.

Condition (c). For the portion of a beam between one end and the first intermediate restraint, account should be taken of the restraint conditions

Table 5.5 Bending strength p_b (N/mm^2) for rolled sections (BS 5950 Part 1 1990 Table 11)

l_{LT}	p_y 245	265	275	325	340	355	415	430	450
30	245	265	275	325	340	355	408	421	438
35	245	265	273	316	328	341	390	402	418
40	238	254	262	302	313	325	371	382	397
45	227	242	250	287	298	309	350	361	374
50	217	231	238	272	282	292	329	338	350
55	206	219	226	257	266	274	307	315	325
60	195	207	213	241	249	257	285	292	300
65	185	196	201	225	232	239	263	269	276
70	174	184	188	210	216	222	242	247	253
75	164	172	176	195	200	205	223	226	231
80	154	161	165	181	186	190	204	208	212
85	144	151	154	168	172	175	188	190	194
90	135	141	144	156	159	162	173	175	178
95	126	131	134	144	147	150	159	161	163
100	118	123	125	134	137	139	147	148	150
105	111	115	117	125	127	129	136	137	139
110	104	107	109	116	118	120	126	127	128
115	97	101	102	108	110	111	117	118	119
120	91	94	96	101	103	104	108	109	111
125	86	89	90	95	96	97	101	102	103
130	81	83	84	89	90	91	94	95	96
135	76	78	79	83	84	85	88	89	90
140	72	74	75	78	79	80	83	84	84
145	68	70	71	74	75	75	78	79	79
150	64	66	67	70	70	71	73	74	75
155	61	62	63	66	66	67	69	70	70
160	58	59	60	62	63	63	65	66	66
165	55	56	57	59	60	60	62	62	63
170	52	53	54	56	56	57	59	59	59
175	50	51	51	53	54	54	56	56	56
180	47	48	49	51	51	51	53	53	53
185	45	46	46	48	49	49	50	50	51
190	43	44	44	46	46	47	48	48	48
195	41	42	42	44	44	44	46	46	46
200	39	40	40	42	42	42	43	44	44
210	36	37	37	38	39	39	40	40	40
220	33	34	34	35	35	36	36	37	37
230	31	31	31	32	33	33	33	34	34
240	29	29	29	30	30	30	31	31	31
250	27	27	27	28	28	28	29	29	29

Table 5.6 Effective length L_E for beams (BS 5950 Part 1 1990 Table 9)

Conditions of restraint at supports		Loading conditions Normal	Destabilizing
Compression flange laterally restrained Beam fully restrained against torsion	Both flanges fully restrained against rotation on plan	0.7 L	0.85 L
	Both flanges partially restrained against rotation on plan	0.85 L	1.0 L
	Both flanges free to rotate on plan	1.0 L	1.2 L
Compression flange laterally unrestrained Both flanges free to rotate on plan	Restraint against torsion provided only by positive connection of bottom flange to supports	1.0 L + 2 D	1.2 L + 2 D
	Restraint against torsion provided only by dead bearing of bottom flange on supports	1.2 L+ 2 D	1.4 L + 2 D

D is the depth of the beam.
L is the span of the beam.

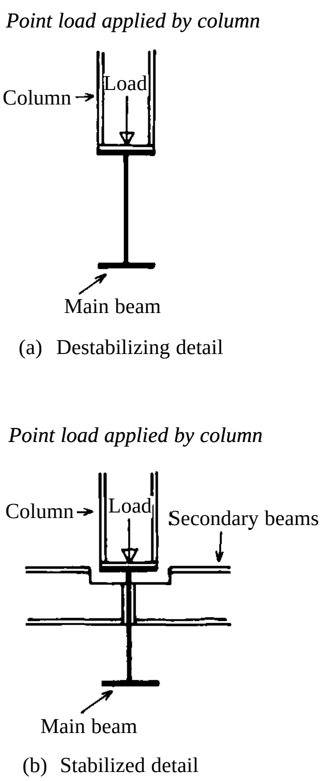


Figure 5.10 *Destabilizing load*

at the support. Therefore the effective length L_E should be taken as the mean of the value given by condition (b) and the value from Table 5.6 elating to the manner of restraint at the support. In both cases, L is taken as the distance between the restrain and the support.

The destabilizing load referred to in the table exists when the member applying the load to the compression flange can move laterally with the beam in question, as illustrated in Figure 5.10a. This may be avoided by the introduction of stabilizing members such as the secondary beams shown in Figure 5.10b.

The slenderness factor v is obtained from BS 5950 Table 14, reproduced here as Table 5.7, using N and l/x, where l is the slenderness, x is the torsional index of the section from section tables, and N is 0.5 for beams with equal flanges.

To check the adequacy of a particular steel beam section, the buckling moment M_b should be compared with the equivalent uniform moment $\bar{M}$:

$$\bar{M} \text{ £ } M_b$$

where $\bar{M} = mM_A$, m is the equivalent uniform moment factor from BS 5950, and M_A is the maximum moment on the member or portion of the member under consideration.

Table 5.7 Slenderness factor *v* for flanged beams of uniform section (BS 5950 Part 1 1990 Table 14)

	Compression / Tension						Compression / Tension				
N / l/x	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
0.5	0.79	0.81	0.84	0.88	0.93	1.00	1.11	1.28	1.57	2.20	12.67
1.0	0.78	0.80	0.83	0.87	0.92	0.99	1.10	1.27	1.53	2.11	6.36
1.5	0.77	0.80	0.82	0.86	0.91	0.97	1.08	1.24	1.48	1.98	4.27
2.0	0.76	0.78	0.81	0.85	0.89	0.96	1.06	1.20	1.42	1.84	3.24
2.5	0.75	0.77	0.80	0.83	0.88	0.93	1.03	1.16	1.35	1.70	2.62
3.0	0.74	0.76	0.78	0.82	0.86	0.91	1.00	1.12	1.29	1.57	2.21
3.5	0.72	0.74	0.77	0.80	0.84	0.89	0.97	1.07	1.22	1.46	1.93
4.0	0.71	0.73	0.75	0.78	0.82	0.86	0.94	1.03	1.16	1.36	1.71
4.5	0.69	0.71	0.73	0.76	0.80	0.84	0.91	0.99	1.11	1.27	1.55
5.0	0.68	0.70	0.72	0.75	0.78	0.82	0.88	0.95	1.05	1.20	1.41
5.5	0.66	0.68	0.70	0.73	0.76	0.79	0.85	0.92	1.01	1.13	1.31
6.0	0.65	0.67	0.69	0.71	0.74	0.77	0.82	0.89	0.97	1.07	1.22
6.5	0.64	0.65	0.67	0.70	0.72	0.75	0.80	0.86	0.93	1.02	1.14
7.0	0.63	0.64	0.66	0.68	0.70	0.73	0.78	0.83	0.89	0.97	1.08
7.5	0.61	0.63	0.65	0.67	0.69	0.72	0.76	0.80	0.86	0.93	1.02
8.0	0.60	0.62	0.63	0.65	0.67	0.70	0.74	0.78	0.83	0.89	0.98
8.5	0.59	0.60	0.62	0.64	0.66	0.68	0.72	0.76	0.80	0.86	0.93
9.0	0.58	0.59	0.61	0.63	0.64	0.67	0.70	0.74	0.78	0.83	0.90
9.5	0.57	0.58	0.60	0.61	0.63	0.65	0.68	0.72	0.76	0.80	0.86
10.0	0.56	0.57	0.59	0.60	0.62	0.64	0.67	0.70	0.74	0.78	0.83
11.0	0.54	0.55	0.57	0.58	0.60	0.61	0.64	0.67	0.70	0.73	0.78
12.0	0.53	0.54	0.55	0.56	0.58	0.59	0.61	0.64	0.66	0.70	0.73
13.0	0.51	0.52	0.53	0.54	0.56	0.57	0.59	0.61	0.64	0.66	0.69
14.0	0.50	0.51	0.52	0.53	0.54	0.55	0.57	0.59	0.61	0.63	0.66
15.0	0.49	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61	0.63
16.0	0.47	0.48	0.49	0.50	0.51	0.52	0.53	0.55	0.57	0.59	0.61
17.0	0.46	0.47	0.48	0.49	0.49	0.50	0.52	0.53	0.55	0.57	0.58
18.0	0.45	0.46	0.47	0.47	0.48	0.49	0.50	0.52	0.53	0.55	0.56
19.0	0.44	0.45	0.46	0.46	0.47	0.48	0.49	0.50	0.52	0.53	0.55
20.0	0.43	0.44	0.45	0.45	0.46	0.47	0.48	0.49	0.50	0.51	0.53

Note 1: For beams with *equal* flanges, $N = 0.5$; for beams with *unequal* flanges refer to clause 4.3.7.5 of BS 5950.

Note 2: *v* should be determined from the general formulae given in clause B.2.5 of BS 5950, on which this table is based: (a) for sections with *lipped* flanges (e.g. gantry girders composed of channel + universal beam); and (b) for intermediate values to the right of the stepped line in the table.

The factors *m* and *n* are interrelated as shown in BS 5950 Table 13, reproduced here as Table 5.8. From this table it can be seen that, when a beam is *not* loaded between points of lateral restraint, *n* is 1.0 and *m* should be obtained from BS 5950 Table 18. The value of *m* depends upon the ratio of the end moments at the points of restraint. If a beam *is* loaded between points of lateral restraint, *m* is 1.0 and *n* is obtained by reference

Table 5.8 Use of *m* and *n* factors for members of uniform section (BS 5950 Part 1 1990 Table 13)

Description		Members *not* subject to destabilizing loads*		Members subject to destabilizing loads*	
		m	*n*	*m*	*n*
Members loaded between adjacent lateral restraints	Sections with equal flanges	1.0	From Tables 15 and 16 of BS 5950	1.0	1.0
	Sections with unequal flanges	1.0	1.0	1.0	1.0
Members not loaded between adjacent lateral restraints	Sections with equal flanges	From Table 18 of BS 5950	1.0	1.0	1.0
	Sections with unequal flanges	1.0	1.0	1.0	1.0
Cantilevers without intermediate lateral restraints		1.0	1.0	1.0	1.0

*See clause 4.3.4 of BS 5950.

to B3 5950 Tables 15 and 16 (Table 16 is cross-referenced with Table 17). Its value depends upon the ratio of the end moments at the points of restraint and the ratio of the larger moment to the mid-span free moment.

Example 5.2

A simply supported steel beam spans 8 m and supports an ultimate central point load of 170 kN from secondary beams, as shown in Figure 5.11. In addition it carries an ultimate UDL of 9 kN resulting from its self-weight. If the beam is only restrained at the load position and the ends, determine a suitable grade 43 section.

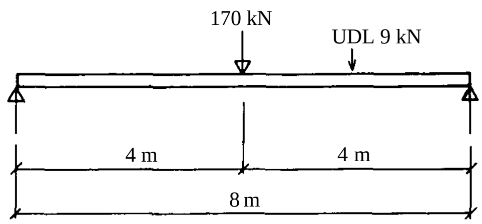


Figure 5.11 *Ultimate load diagram*

The maximum ultimate moment is given by

$$M_A = \frac{WL}{4} + \frac{WL}{8} = \frac{170 \times 8}{4} + \frac{9 \times 8}{8} = 340 + 9 = 349 \text{ kN m}$$

Since the beam is laterally unrestrained it is necessary to select a trial section for checking: try 457 × 152 × 74 kg/m UB (S_x = 1620 cm^3). The moment capacity of this section when the beam is subject to low shear is given by $M_{cx} = p_y S_x$, where p_y is 265 N/mm^2 since T is greater than 16 mm. Thus

$$M_{cx} = p_y S_x = 265 \times 1620 \times 10^3 = 429.3 \times 10^6 \; N\,mm = 429.3 \; kN\,m > 349 \; kN\,m$$

This is adequate.

The lateral torsional buckling resistance is checked in the following manner:

$$\bar{M} = mM_A \text{ £ } M_b = p_b S_x$$

The self-weight UDL of 9 kN is relatively insignificant, and it is therefore satisfactory to consider the beam to be not loaded between restraints. By reference to Table 5.8, for members that are not subject to destabilizing loads, *n* is 1.0 and *m* should be obtained from BS 5950 Table 18.

The values of *m* in Table 18 depend upon *b* , which is the ratio of the smaller end moment to the larger end moment for the unrestrained length being considered. In this example the unrestrained length is the distance from a support to the central point load. The bending moment diagram for this length is shown in Figure 5.12.

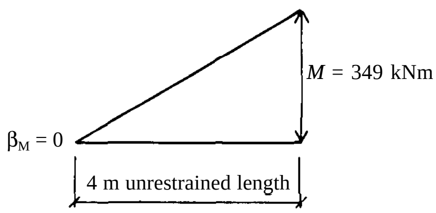


Figure 5.12 *Equivalent bending moment diagram for the unrestrained length*

It can be seen from this diagram that the end moment for a simply supported beam is zero. Hence

$$b = \frac{\text{smaller end moment}}{\text{larger end moment}} = \frac{0}{349} = 0$$

Therefore the value of *m* from BS 5950 Table 18 is 0.57.

It should be appreciated that if the central point load was from a column and there were no lateral beams at that point, then a destabilizing load condition would exist. In such a case both *m* and *n*, from Table 5.8, would be 1.0.

Equivalent uniform moment $\bar{M} = mM_A = 0.57 \times 349 = 198.93$ kN m

Buckling resistance moment $M_b = p_b S_x$

The bending strength p_b has to be obtained from Table 5.5 in relation to p_y and l_{LT}. We have $p_y = 265$ N/mm^2 and

$$l_{LT} = nuvl$$

where $n = 1.0$, $u = 0.87$ from section tables, and $l = L_E/r_y$. In this instance L_E = 1.0 *L* from Table 5.6, where *L* is the distance between restraints, and $r_y = 3.26$ cm $= 3.26 \times 10$ mm from section tables. Thus

$$l = \frac{1.0 \times 4000}{3.26 \times 10} = 122.7$$

Now $x = 30$ from section tables. Hence $l/x = 122.7/30 = 4.09$. and $v = 0.856$ by interpolation from Table 5.7. Hence

$$\lambda_{LT} = nuv\lambda = 1.0 \times 0.87 \times 0.856 \times 122.7 = 91.38$$

Therefore $p_b = 138.24$ N/mm² by interpolation from Table 5.5. Thus finally

$$\begin{aligned} M_b &= p_b S_x = 138.24 \times 1620 \times 10^3 \\ &= 223.95 \times 10^6 \text{ N mm} = 223.95 \text{ kN m} > 198.93 \text{ kN m} \end{aligned}$$

That is, $\bar{M} < M_b$. Therefore the lateral torsional buckling resistance of the section is adequate. In conclusion:

Adopt 457 × 152 × 74 kg/m UB.

The *Steelwork Design Guide* produced by the Steel Construction Institute also contains tables giving both the buckling resistance moment M_b and the moment capacity M_{cx} for the entire range of rolled sections. A typical example of a number of UB sections is reproduced here as Table 5.9. From the table it can be seen that for the 457 × 152 × 74 kg/m UB section that we have just checked, the relevant moment values are as follows:

$M_{cx} = 429$ kN m; and $M_b = 223$ kN m when n is 1.0 and the effective length is 4.0 m. By using these tables the amount of calculation is significantly reduced, and they are therefore a particularly useful design aid for checking beams.

Example 5.3

If the beam in Example 5.2 were to be loaded between lateral restraints as shown in Figure 5.13, what size of grade 43 section would be required?

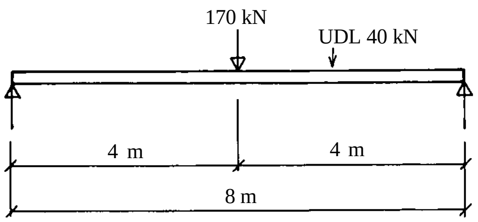


Figure 5.13 *Ultimate load diagram*

The maximum ultimate moment at mid-span is given by

$$M_A = \frac{WL}{4} + \frac{WL}{8} = \frac{170 \times 8}{4} + \frac{40 \times 8}{8} = 340 + 40 = 380 \text{ kN m}$$

It is necessary to select a trial section for checking: try 457 × 191 × 82 kg/m UB ($S_x = 1830$ cm³). Thus

$$M_{cx} = p_y S_x = 275 \times 1830 \times 10^3 = 503.25 \times 10^6 \text{ N mm} = 503.25 \text{ kN m} > 380 \text{ kN m}$$

This is adequate.

Table 5.9 Universal beams subject to bending, steel grade 43: buckling resistance moment M_b(kN m) (abstracted from the *Steelwork Design Guide to BS 5950: Part I*, published by the Steel Construction Institute)

Designation serial size: mass/metre and capacity	Slenderness correction factor n	Effective length L_E 2.0	2.5	3.0	3.5	4.0	4.5	5.0	6.0	7.0	8.0	9.0	10.0	11.0
457 × 191 × 82	0.4	503	503	503	503	503	503	496	472	451	431	413	395	379
M_{cx} = 503	0.6	503	503	500	478	457	436	417	379	346	317	291	269	249
Plastic	0.8	503	480	449	419	389	361	335	289	252	223	199	180	164
	1.0	478	437	396	357	321	289	261	217	184	159	140	126	114
457 × 191 × 74	0.4	456	456	456	456	456	456	446	424	403	384	366	349	333
M_{cx} = 456	0.6	456	456	451	430	410	391	372	337	305	277	253	232	214
Plastic	0.8	456	433	404	375	348	321	296	253	219	192	171	154	140
	1.0	431	393	355	319	285	255	230	189	159	137	120	107	96
457 × 191 × 67	0.4	404	404	404	404	404	402	391	370	350	332	314	298	283
M_{cx} = 404	0.6	404	404	397	378	359	341	323	290	260	234	212	194	178
Plastic	0.8	404	381	354	328	302	277	254	215	184	160	142	127	114
	1.0	380	345	310	277	246	219	195	159	132	113	98	87	78
457 × 152 × 82	0.4	477	477	477	477	475	462	450	427	407	388	370	0	0
M_{cx} = 477	0.6	477	471	447	424	402	381	362	327	297	272	250	0	0
Plastic	0.8	457	422	388	356	326	300	277	238	208	185	167	0	0
	1.0	416	370	327	290	257	231	208	174	149	131	116	0	0
457 × 152 × 74	0.4	429	429	429	429	423	411	399	377	357	339	322	0	0
M_{cx} = 429	0.6	429	421	398	376	355	335	317	284	256	232	212	0	0
Plastic	0.8	409	375	343	313	285	260	239	204	177	156	140	0	0
	1.0	371	328	288	252	223	198	178	147	125	109	97	0	0
457 × 152 × 67	0.4	396	396	396	396	384	372	360	338	318	299	283	0	0
M_{cx} = 396	0.6	396	383	361	339	318	299	280	247	220	198	179	0	0
Plastic	0.8	372	340	308	278	251	227	207	174	149	130	116	0	0
	1.0	336	294	255	221	193	170	152	124	105	90	79	0	0
457 × 152 × 60	0.4	352	352	352	351	339	328	317	296	276	259	243	0	0
M_{cx} = 352	0.6	352	340	319	299	280	261	244	213	188	167	151	0	0
Plastic	0.8	330	301	272	244	219	197	178	148	126	109	96	0	0
	1.0	298	260	224	193	168	147	130	105	87	75	66	0	0
457 × 152 × 52	0.4	300	300	300	295	284	274	263	243	225	208	194	0	0
M_{cx} = 300	0.6	300	286	267	249	231	214	198	170	148	130	116	0	0
Plastic	0.8	278	251	225	200	178	158	142	116	97	83	73	0	0
	1.0	249	215	183	156	134	116	102	81	67	57	50	0	0
406 × 178 × 74	0.4	412	412	412	412	412	412	404	386	369	354	339	326	313
M_{cx} = 412	0.6	412	412	405	387	370	354	338	309	283	260	240	223	208
Plastic	0.8	412	388	362	337	313	291	270	235	206	183	165	150	137
	1.0	385	351	317	286	257	232	210	175	150	131	116	105	95
406 × 178 × 67	0.4	371	371	371	371	371	370	360	343	327	312	298	285	273
M_{cx} = 371	0.6	371	371	363	346	330	314	299	271	246	225	206	190	176
Plastic	0.8	371	347	323	300	277	256	236	203	177	156	139	126	115
	1.0	345	313	282	252	226	202	182	151	128	111	97	87	79

M_b is obtained using an equivalent slenderness = $nuvL_e/r_y$.
Values have not been given for values of slenderness greater than 300.
The section classification given applies to members subject to bending only.

Check lateral torsional buckling:

$$\bar{M} = mM_A \leq M_b = p_b S_x$$

The magnitude of the UDL in this example is significant, and it will therefore be necessary to consider the beam to be loaded between lateral restraints. By reference to Table 5.8, for members not subject to destabilizing loads, m is 1.0 and n should be obtained from BS 5950 Table 16, which is cross-referenced with Table 17.

First we have

$$\bar{M} = mM_A = 1.0 \times 380 = 380 \text{ kN m}$$

We now need to find M_b. The bending strength p_b has to be obtained from Table 5.5 in relation to p_y and l_{LT}. We have $p_y = 275$ N/mm^2 and

$$l_{LT} = nuvl$$

The slenderness correction factor n is obtained from BS 5950 Table 16 in relation to the ratios g and $ß$ for the length of beam between lateral restraints. In this instance that length would be from a support to the central point load. The ratios g and $ß$ are obtained as follows. First, $g = M/M_o$. The larger end moment M = 380 kN m. M_o is the mid-span moment on a simply supported span equal to the unrestrained length, that is $M_o = WL/8$. The UDL on unrestrained length is W = 40/2 = 20 kN, and the unrestrained length L = span/2 = 4 m. Hence

$$M_o = \frac{WL}{8} = \frac{20 \times 4}{8} = 10 \text{ kNm}$$

The equivalent bending moment diagram, for the unrestrained length, corresponding to these values is shown in Figure 5.14. Thus

$$g = \frac{M}{M_o} = \frac{380}{10} = 38$$

Secondly,

$$ß = \frac{\text{smaller end moment}}{\text{larger end moment}} = \frac{0}{380} = 0$$

Therefore, by interpolation from BS 5950 Table 16, $n = 0.782$.

From section tables, $u = 0.877$.

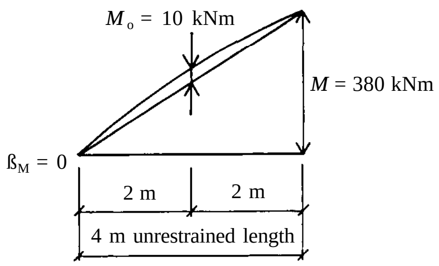


Figure 5.14 *Equivalent bending moment diagram for the unrestrained length*

Next $l = L_E/r_y$, where $L_E = 1.0\,L$ in this instance from Table 5.6; L is the distance between restraints; and $r_y = 4.23$ cm from section tables, that is 4.23×10 mm. Thus

$$l = \frac{L_E}{r_y} = \frac{1.0 \times 4000}{4.23 \times 10} = 94.56$$

Here $x = 30.9$ from section tables. Therefore $l/x = 94.56/30.9 = 3.06$, and so $v = 0.91$ from Table 5.7.

Finally, therefore,

$$l_{LT} = nuvl = 0.782 \times 0.877 \times 0.91 \times 94.56 = 59$$

Using the values of p_y and l_{LT}, $p_b = 215.6\ \text{N/mm}^2$ by interpolation from Table 5.5. In conclusion,

$$M_b = p_b S_x = 215.6 \times 1830 \times 10^3 = 394.5 \times 10^6\ \text{N mm} = 394.5\ \text{kN m} > 380\ \text{kN m}$$

Thus $\bar{M} < M_b$, and therefore the lateral torsional buckling resistance of the section is adequate.

Adopt 457 × 191 × 82 kg/m UB.

The M_{cx} and M_b values that we have calculated may be compared with those tabulated by the Steel Construction Institute for a 457 × 191 × 82 kg/m UB. From Table 5.9, $M_{cx} = 503$ kN m, and $M_b = 389$ kN m when n is 0.8 and the effective length is 4.0.

Laterally unrestrained beams, conservative approach

The suitability of laterally unrestrained UB, UC and RSJ sections may be checked, if desired, using a conservative approach. It should be appreciated that being conservative the design will not be as economic as that given by the rigorous approach; consequently beam sections that are proved to be adequate using the rigorous approach may occasionally prove inadequate using the conservative approach. However, it does have the advantage that members either loaded or unloaded between restraints are checked using one expression.

In the conservative approach the maximum moment M_x occurring between lateral restraints must not exceed the buckling resistance moment M_b:

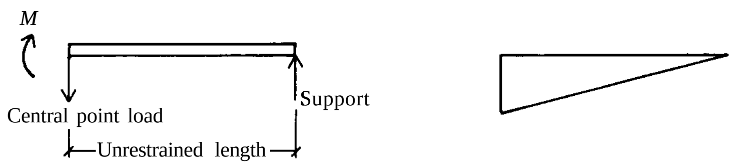
$$M_x \leq M_b$$

The buckling resistance moment is given by the expression

$$M_b = p_b S_x$$

For the conservative approach, p_b is obtained from the appropriate part of Table 19 a–d of BS 5950 in relation to l and x, the choice depending on the design strength p_y of the steel.

Loads occurring between restraints may be taken into account by multiplying the effective length by a slenderness correction factor n obtained either from BS 5950 Table 13 (reproduced earlier as Table 5.8) or alternatively from BS 5950 Table 20, except for destabilizing loads when it should be taken as 1.0. It is important to understand that the reactions shown on the diagrams in Table 20 are the lateral restraints and not just the beam supports. Therefore for a simply supported beam with a central point load providing lateral restraint, the relevant Table 20 diagram would be as shown in Figure 5.15. The corresponding value of n would then be 0.77.



(a) Portion of beam between lateral restraints

(b) Corresponding bending moment diagram

Figure 5.15 *Conservative approach slenderness correction factor diagrams for a simply supported beam restrained at mid-span*

Thus the minor axis slenderness ratio is given by

$$l = \frac{nL_E}{r_y}$$

where *n* is the slenderness correction factor either from BS 5950 Table 13 or Table 20, L_E is the effective unrestrained length of the beam, and r_y is the radius of gyration of the section about its minor axis, found from section tables. The torsional index *x* of the section is taken from section tables.

For those who are familiar with BS 449, this approach is similar to the use of Table 3 in that standard, which was related to the *l*/*r* and *D*/*T* ratios of the section.

Example 5.4

Check the beam section selected in Example 5.3, using the conservative approach.

The maximum ultimate moment M_x = 380 kN n at midspan. Check 457 × 191 × 82 kg/m UB (S_x = 1830 cm^3). T = 16mm; hence p_y = 275 N/mm^2. Thus

$$M_{cx} = p_y S_x = 275 \times 1830 \times 10^3 = 503.25 \times 10^6 \text{ N mm} = 503.25 \text{ kN m} > 380 \text{ kN m}$$

This is satisfactory:

Check lateral torsional buckling, that is show

$$M_x \leq M_b = p_b S_x$$

For the conservative approach, p_b is obtained from BS 5950 Table 19b when p_y is 275 N/mm^2, using l and *x*. The slenderness correction factor *n* obtained from BS 5950 Table 20 is 0.77. Then

$$l = \frac{nL_E}{r_y} = \frac{0.77 \times 4000}{4.23 \times 10} = 72.8$$

Now x = 30.9. Thus p_b = 210 N/mm^2 by interpolation from BS 5950 Table 19b. So

$$M_b = p_b S_x = 210 \times 1830 \times 10^3 = 384.3 \times 10^6 \text{ N mm} = 384.3 \text{ kN m} > 380 \text{ kN m}$$

Therefore $M_x < M_b$, and so the lateral torsional buckling resistance of the section is adequate.

5.10.4 Shear ULS

The shear resistance of a beam is checked by ensuring that the ultimate shear force F_v does not exceed the shear capacity P_v of the section at the point under consideration:

$$F_v \leq P_v$$

where

F_v ultimate shear force at point under consideration
P_v shear capacity of section: $P_v = 0.6 p_y A_v$
p_y design strength of steel, given in Table 5.1.
A_v area of section resisting shear: $A_v = tD$ for rolled sections, as shown in Figure 5.16
t total web thickness, from section tables
D overall depth of section, from section tables

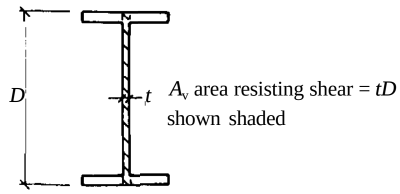


Figure 5.16 *Area of a rolled section resisting shear*

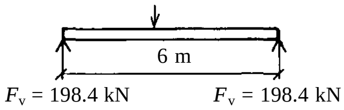
It is recommended in BS 5950 that the combination of maximum moment and coexistent shear, and the combination of maximum shear and coexistent moment, should be checked. The moment capacity of plastic and compact beam sections is reduced when high shear loads occur. A high shear load is said to exist when the ultimate shear force exceeds 0.6 times the shear capacity of the section, that is when $F_v > 0.6\, P_v$. However, as mentioned in Example 5.1, this is not usually a problem except for heavily loaded short span beams.

When the depth to thickness ratio d/t of a web exceeds $63e$, where $e = (275/p_y)^{1/2}$ as previously referred to in Table 5.4, the web should be checked for shear buckling. This does not apply to any of the standard rolled sections that are available, but it may apply to plate girders made with thin plates.

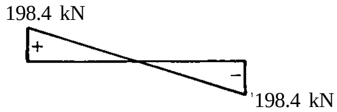
It should be appreciated that, if necessary, the web of a beam may be strengthened locally to resist shear by the introduction of stiffeners, designed in accordance with the recommendations given in BS 5950.

Example 5.5

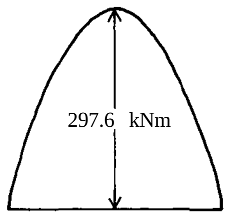
Check the shear capacity of the beam that was designed for bending in Example 5.1. The loading, shear force and bending moment diagrams for the beam are shown in Figure 5.17.



(a) Ultimate load diagram



(b) Shear force diagram



(c) Bending moment diagram

Figure 5.17 *Beam diagrams for ultimate loads*

The section selected to resist bending was a 457 × 152 × 60 kg/m UB, for which the relevant properties for checking shear, from Table 5.2, are t = 8.0 mm and D = 454.7 mm. Beam sections should normally be checked for the combination of maximum moment and coexistent shear, and the combination of maximum shear and coexistent moment. However, since the beam in this instance only carried a UDL the shear is zero at the point of maximum moment. Therefore it will only be necessary to check the section at the support where the maximum shear occurs and the coexistent moment is zero.

Ultimate shear at support $F_v = 198.4$ kN

Shear capacity of section $P_v = 0.6\,p_y A_v = 0.6\,p_y tD$

$$= 0.6 \times 275 \times 8 \times 454.7 = 600\,204 \text{ N}$$

$$= 600 \text{ kN} > 198 \text{ kN}$$

That is $F_v < P_v$, and therefore the section is adequate in shear.

Example 5.6

Check the shear capacity of the beam that was designed for bending in Example 5.2. The loading, shear force and bending moment diagrams for the beam are shown in Figure 5.18.

The section selected to resist bending was a 457 × 152 × 74 kg/m UB, for which the relevant properties for checking shear, from Table 5.2, are t = 9.9 mm and D = 461.3 mm. In addition it should be noted that the flange thickness T of this section

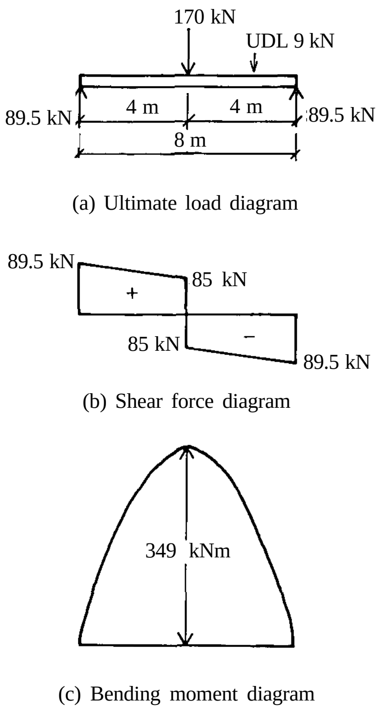


Figure 5.18 *Beam diagrams for ultimate loads*

is greater than 16 mm, and therefore the reduced p_y value of 265 N/mm^2 should be used in the calculations.

This beam will be checked for the combination of maximum moment and co-existent shear, and the combination of maximum shear and co-existent moment.

Maximum moment and coexistent shear at midspan

Ultimate shear at midspan F_v = 85 kN; M = 349 kN m

Shear capacity of section $P_v = 0.6\,p_y tD$

$$= 0.6 \times 265 \times 9.9 \times 461.3 = 726\ 132\ \text{N}$$
$$= 726\ \text{kN} > 85\ \text{kN}$$

Furthermore $0.6\,P_v = 0.6 \times 726 = 435.6$ kN. Therefore

$$F_v = 85\ \text{kN} < 0.6\text{P}_v = 435.6\ \text{kN}$$

Hence the shear load is low and no reduction to the moment capacity calculated earlier is necessary because of shear.

Maximum shear and co-existent moment

Ultimate shear at support F_v = 89.5 kN $M = 0$

Shear capacity of section P_v = 726 kN > 89.5 kN

That is $F_v < P_v$, and therefore the section is adequate in shear.

5.10.5 Deflection SLS

The deflection limits for steel beams are given in BS 5950 Table 5. For beams carrying plaster or other brittle finish the limit is span/360, and for all other beams is span/200. That is,

$$\text{Permissible deflection } d_p = \frac{\text{span}}{360} \quad \text{or} \quad \frac{\text{span}}{200}$$

It should be appreciated that these are only recommended limits and in certain circumstances more stringent limits may be appropriate. For example the deflection of beams supporting glazing or door gear may be critical to the performance of such items, in which case a limit of span/500 may be more realistic.

The actual deflection produced by the unfactored imposed loads alone should be compared with these limits. This is calculated using the formula relevant to the applied loading. For example,

$$\text{Actual deflection due to a UDL } d_a = \frac{5}{384}\frac{WL^3}{EI}$$

$$\text{Actual deflection due to a central point load } d_a = \frac{1}{48}\frac{WL^3}{EI}$$

where, in relation to steel sections, $E = 205 \text{ kN/mm}^2 = 205 \times 10^3 \text{ N/mm}^2$, and I is the second moment of area of the section about its major x–x axis, found from section tables. That is,

$$d_a \leq d_p$$

Example 5.7

Check the deflection of the beam that was designed for bending in Example 5.3 if the unfactored imposed loads are as shown in Figure 5.19.

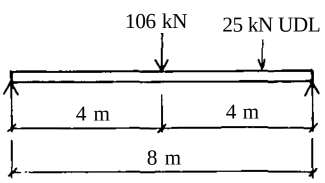


Figure 5.19 *Unfactored imposed loads*

The section selected to resist bending was a 457 × 191 × 82 kg/m UB, for which the second moment of area I_x is 37 100 cm^4. The deflection limit is given by

$$d_p = \frac{\text{span}}{360} = \frac{8000}{360} = 22.22 \text{ mm}$$

The actual deflection is

$$\begin{aligned} d_a &= \frac{5}{384}\frac{WL^3}{EI} + \frac{1}{48}\frac{WL^3}{EI} \\ &= \frac{5}{384} \times \frac{25 \times 10^3 \times 8000^3}{205 \times 10^3 \times 37\,100 \times 10^4} \\ &\quad + \frac{1}{48} \times \frac{106 \times 10^3 \times 8000^3}{205 \times 10^3 \times 37\,100 \times 10^4} \\ &= 2.19 + 14.87 = 17.06 \text{ mm} < 22.22 \text{ mm} \end{aligned}$$

That is $d_a < d_p$, and therefore the section is adequate in deflection.

5.10.6 Web buckling resistance

When a concentrated load, such as the reaction, is transmitted through the flange of a beam to the web, it may cause the web to buckle. In resisting such buckling the web of the beam behaves as a strut. The length of web concerned is determined on the assumption that the load is dispersed at 45° from the edge of stiff bearing to the neutral axis of the beam, as shown in Figure 5.20. The buckling resistance P_w of the unstiffened web is calculated from the following expression:

$$P_w = (b_1 + n_1)tp_c$$

where

b_1 stiff bearing length
n_1 length obtained by dispersion through half the depth of the section
t web thickness, from section tables
p_c compressive strength of the steel

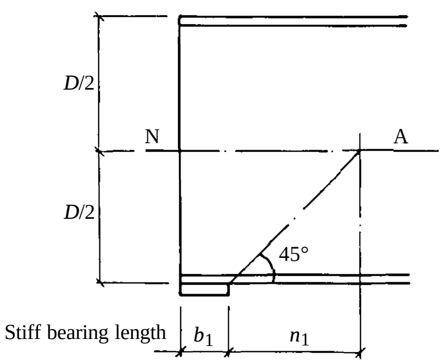


Figure 5.20 *Web buckling resistance: load dispersal*

The compressive strength p_c of the steel should be obtained from BS 5950 Table 27c in relation to the ultimate design strength of the steel and the web slenderness l.

When the beam flange through which the load is applied is restrained against rotation relative to the web and against lateral movement relative to the other flange, then the slenderness is given by the following expression from BS 5950:

$$l = 2.5\frac{d}{t}$$

Should these conditions not apply, then the slenderness may conservatively be obtained using the following expression:

$$l = 3.46\frac{d}{t}$$

Example 5.8

Check the web buckling capacity of the beam that was designed for bending in Example 5.1. It may be assumed that the beam is supported on a stiff bearing length of 75 mm as indicated in Figure 5.21.

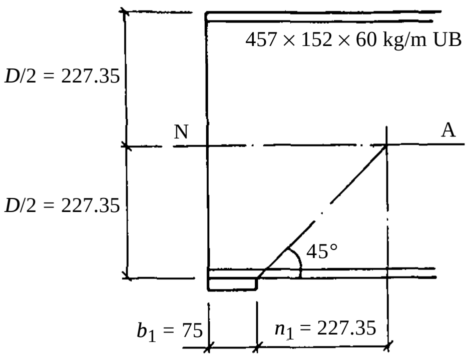


Figure 5.21 *Web buckling check dimensions*

From the loading diagram for this beam, shown in Figure 5.17, the maximum ultimate reaction is 198.4 kN.

The section selected to resist bending was a 457 × 152 × 60 kg/m UB, for which the relevant properties for checking web buckling, from Table 5.2, are as follows:

$$D = 454.7\,\text{mm} \qquad \frac{D}{2} = \frac{454.7}{2} = 227.35\,\text{mm}$$

$$\frac{d}{t} = 51.00 \qquad t = 8.0\,\text{mm}$$

With both flanges restrained,

$$l = 2.5\frac{d}{t} = 2.5 \times 51 = 127.5$$

Also $p_y = 275$ N/mm^2. Thus by interpolation from BS 5950 Table 27c, $p_c = 88.5$ N/mm^2.

The stiff bearing length $b_1 = 75$ mm, and $n_1 = D/2 = 227.35$ mm. Hence

$$P_w = (b_1 + n_1)tp_c = (75 + 227.35)8 \times 88.5 = 214\,064\ \text{N} = 214\ \text{kN} > 198.4\ \text{kN}$$

Thus the buckling resistance of the unstiffened web is greater than the maximum reaction, and therefore the web does not require stiffening to resist buckling.

5.10.7 Web bearing resistance

The web bearing resistance of a beam is the ability of its web to resist crushing induced by concentrated loads such as the reactions. These are considered to be dispersed through the flange at a slope of 1:2.5 to the point where the beam flange joins the web, that being the critical position for web bearing (see Figure 5.22).

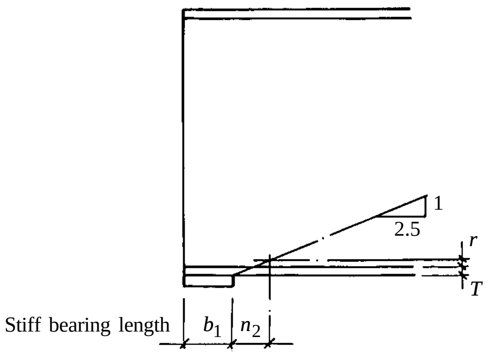


Figure 5.22 *Web bearing resistance: load dispersal*

The ultimate web bearing capacity P_{crip} of a beam is given by the following expression:

$$P_{\text{crip}} = (b_1 + n_2)tp_{\text{yw}}$$

where

b_1 stiff bearing length

n_2 length obtained by dispersion at a slope of 1:2.5 through the flange to the flange to web connection: $n_2 = 2.5(r + T)$

r root radius of the beam, from section tables

T beam flange thickness, from section tables

p_{yw} design strength of the web: $p_{\text{yw}} = p_{\text{y}}$

Example 5.9

Check the web bearing capacity of the beam that was designed for bending in Example 5.1. It may be assumed that the beam is supported on a stiff bearing length of 75 mm, as indicated in Figure 5.23.

From the loading diagram for this beam shown in Figure 5.17, the maximum ultimate reaction is 198.4 kN.

The section selected to resist bending was a 457 × 152 × 60 kg/m UB, for which the relevant properties for checking web bearing, from Table 5.2, are as follows:

$$r = 10.2\,\text{mm} \qquad T = 13.3\,\text{mm} \qquad t = 8.0\,\text{mm}$$

$$\text{Stiff bearing length } b_1 = 75\,\text{mm}$$

$$n_2 = 2.5(r + T) = 2.5(10.2 + 13.3) = 58.75\,\text{mm}$$

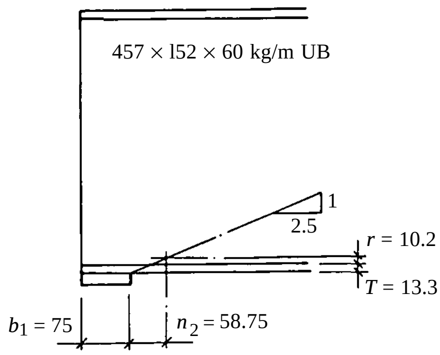


Figure 5.23 *Web bearing check dimensions*

Hence

$$P_{\text{crip}} = (b_1 + n_2)tp_{\text{yw}} = (75 + 58.75)8 \times 275 = 294\,250\ \text{N} = 294\ \text{kN} > 198.4\ \text{kN}$$

Thus the bearing resistance of the unstiffened web is greater than the maximum reaction, and therefore the web does not require stiffening to resist crushing due to bearing.

5.10.8 Design summary for steel beams

Having examined the various aspects that can influence the design of steel beams, the general procedure when using grade 43 rolled sections may be summarized as follows.

Bending

(a) Decide if the beam will be laterally restrained or laterally unrestrained.

(b) If the beam is laterally restrained, ensure that the moment capacity M_{cx} of the section is greater than the applied ultimate moment M_{u}:

$$M_{\text{cx}} = p_{\text{y}}S_{\text{x}} \geq M_{\text{u}}$$

(c) If the beam is laterally unrestrained, the lateral torsional buckling resistance of the section will have to be checked. This may be done using a rigorous approach or a conservative approach. In both methods, account should be taken of any loading between restraints.

(i) Using the rigorous approach, ensure that the applied equivalent uniform moment $\bar{M}$ is less than the buckling resistance moment M_{b} of the section:

$$\bar{M} = mM_{\text{A}} \leq M_{\text{b}} = p_{\text{b}}S_{\text{x}}$$

(ii) Using the conservative approach, ensure that the maximum moment M_{x} occurring between lateral restraints does not exceed the buckling resistance moment M_{b} of the section:

$$M_{\text{x}} \leq M_{\text{b}} = p_{\text{b}}S_{\text{x}}$$

Shear

Both the combination of maximum moment and coexistent shear, and the combination of maximum shear and coexistent moment, should be checked.

The shear resistance of a beam is checked by ensuring that the ultimate shear force F_v does not exceed the shear capacity P_v of the section at the point under consideration:

$$F_v \leq P_v$$

It should be noted that the moment capacity of plastic and compact beam sections must be reduced when high shear loads occur. However, this is not usually a problem except for heavily loaded short span beams.

A high shear load condition exists when

$$F_v > 0.6P_v$$

Deflection

The deflection requirement of a beam is checked by comparing the actual deflection produced by the unfactored imposed loads with the recommended limits given in BS 5950 Table 5:

Actual deflection < recommended deflection limit

Web buckling

The web buckling resistance of an unstiffened web must be greater than any concentrated load that may be applied.

Web bearing

The web bearing resistance of an unstiffened web must be greater than any concentrated load that may be applied.

It should be appreciated that the requirements for web buckling and bearing are not usually critical under normal loading conditions. Furthermore, they can if necessary be catered for by the inclusion of suitably designed web stiffeners.

Before leaving the topic of beams, let us look at a further example illustrating the complete design of a laterally unrestrained beam using the rigorous approach.

Example 5.10

The simply supported beam shown in Figure 5.24 is laterally restrained at the ends and at the points of load application. For the loads given below, determine the size of grade 43 section required.

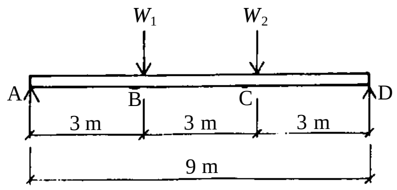


Figure 5.24 *Simply supported beam*

Specified dead loads:
Point load W_{1d} = 30 kN; point load W_{2d} = 20 kN
Self-weight = 1 kN/m; SW UDL = 1×9 = 9 kN

Specified imposed loads:
Point load W_{1i} = 50 kN; point load W_{2i} = 30 kN

Ultimate design loads:
$W_1 = g_f W_{1d} + g_f W_{1i} = 1.4 \times 30 + 1.6 \times 50 = 42 + 80 = 122$ kN
$W_2 = g_f W_{2d} + g_f W_{2i} = 1.4 \times 20 + 1.6 \times 30 = 28 + 48 = 76$ kN
SW UDL = 1.4×9 = 12.6 kN

The ultimate design load diagram and the corresponding shear force and bending moment diagrams are shown in Figure 5.25. Since the loading is not symmetrical, the reactions and moments are calculated from first principles.

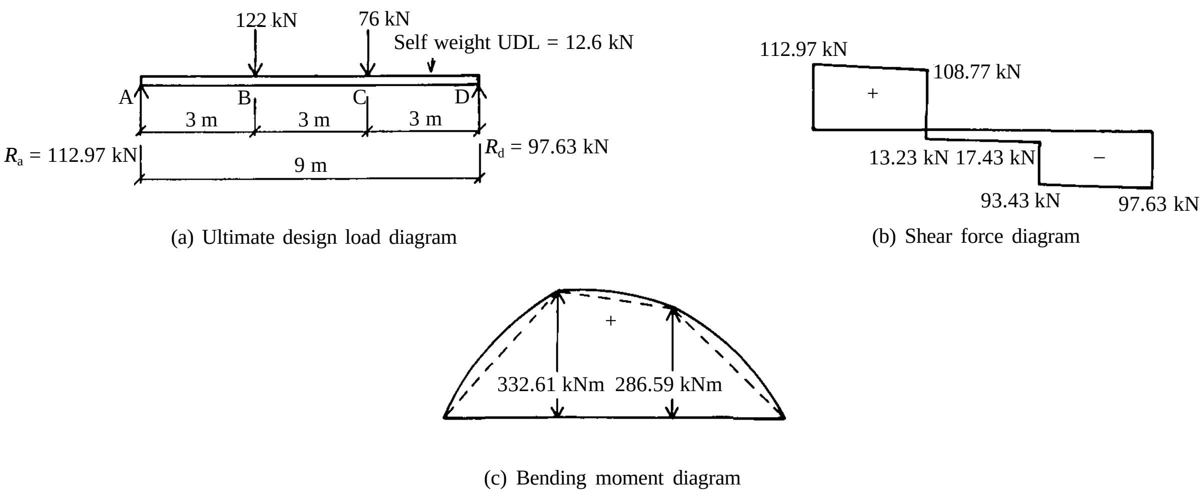


Figure 5.25 *Beam diagrams for ultimate loads*

For the reactions, take moments about D:

$$9R_a = (122 \times 6) + (76 \times 3) + (12.6 \times 4.5) = 732 + 228 + 56.7 = 1016.7$$
$$R_a = 1016.7/9 = 112.97 \text{ kN}$$
$$R_d = (122 + 76 + 12.6) - 112.97 = 97.63 \text{ kN}$$

$$\text{Ultimate moment at B} = 112.97 \times 3 - \frac{12.6}{9} \times \frac{3^2}{2} = 332.61 \text{ kN m}$$

$$\text{Ultimate moment at C} = 97.63 \times 3 - \frac{12.6}{9} \times \frac{3^2}{2} = 286.59 \text{ kN m}$$

Since the beam is not fully restrained laterally, the buckling resistance moment M_b of the section needs to be checked in comparison with the applied equivalent uniform moment $\bar{M}$ to ensure that

$$\bar{M} = mM_A \leq M_b = p_b S_x$$

By reference to the bending moment diagram shown in Figure 5.25c, the critical unrestrained length will be BC where the maximum moment occurs.

The self-weight UDL is relatively insignificant and it is therefore satisfactory to consider the beam to be unloaded between restraints. Hence n is 1.0 and m is obtained from BS 5950 Table 18. We have

$$ß = \frac{\text{smaller end moment}}{\text{larger end moment}} = \frac{M \text{ at C}}{M \text{ at B}} = \frac{286.59}{332.61} = 0.86$$

Therefore by interpolation from Table 18, $m = 0.93$. The maximum moment on length BC is $M_A = M$ at B = 332.61 kN m. Hence

$$\bar{M} = mM_A = 0.93 \times 332.61 = 309.33 \, kN\, m$$

The effective length L_E of BC is 3.0 m.

By reference to Table 5.9, reproduced from the Steel Construction Institute design guide, a 457 × 191 × 74 kg/m UB has a buckling resistance moment of 355 kN m when n is 1 and L_E is 3.0 m. Therefore let us check this section in bending, shear and deflection.

The relevant properties for the section from tables are as follows:

Plastic modulus $S_x = 1660 \text{ cm}^3$

$D = 457.2$ mm $t = 9.1$ mm $T = 14.5$ mm $d = 407.9$ mm

Section classification: plastic

Since $T = 14.5 \text{ mm} < 16 \text{ mm}$, $p_y = 275 \text{ N/mm}^2$.

Check the section for combined moment and shear as follows.

Maximum moment and coexistent shear at B

Ultimate shear at B is $F_v = 108.77$ kN; $M = 332.61$ kN m

Shear capacity of section is $P_v = 0.6\, p_y t D$
$= 0.6 \times 275 \times 9.1 \times 457.2 = 686\,486 \, N$
$= 686 \text{ kN} > 108.77 \text{ kN}$

This is satisfactory. Furthermore, $0.6P_v = 0.6 \times 686 = 412$ kN. Therefore

$$F_v = 108.77 \text{ kN} < 0.6P_v = 412 \text{ kN}$$

Hence the shear load is low and the moment capacity is as follows:

$$M_{cx} = p_y S_x = 275 \times 1660 \times 10^3 = 456.5 \times 10^6 \text{ N mm} = 456.5 \text{ kN m} > 332.61 \text{ kN m}$$

Maximum shear and coexistent moment at A

Ultimate shear at A is $F_v = 112.97$ kN; $M = 0$

P_v is again 686 kN > 112.97 kN

Buckling resistance

The lateral torsional buckling resistance has already been satisfied by selecting a section from Table 5.9 with a buckling resistance moment M_b greater than the equivalent uniform moment $\bar{M}$. However, the method of calculating the buckling resistance moment in accordance with BS 5950 will be included here for reference.

The buckling resistance moment of the section is given by

$$M_b = p_b S_x$$

The bending strength p_b is obtained from Table 5.5 in relation to p_y and l_{LT}. We have $p_y = 275$ N/mm^2 and

$$l_{LT} = nuvl$$

Now $n = 1.0$, and $u = 0.876$ from section tables. Next $l = L_E/r_y$, where $L_E = 1.0$ L in this instance from Table 5.6; L is the distance BC between restraints; and r_y = 4.19 cm = 4.19 × 10 mm from section tables. Thus

$$l = \frac{L_E}{r_y} = \frac{1.0 \times 3000}{4.19 \times 10} = 71.6$$

Here $x = 33.9$ from section tables. Thus $l/x = 71.6/33.9 = 2.11$, and so $v = 0.95$ by interpolation from Table 5.7.

Finally, therefore,

$$l_{LT} = nuvl = 1.0 \times 0.876 \times 0.95 \times 71.6 = 59.6$$

Using the values of p_y and l_{LT}, $p_b = 214$ N/mm^2 from Table 5.5. In conclusion,

$$M_b = p_b S_x = 214 \times 1660 \times 10^3 = 355.2 \times 10^6 \text{ N mm} = 355.2 \text{ kN m} > 309.33 \text{ kN m}$$

Thus $\bar{M} < M_b$, and therefore the lateral torsional buckling resistance of the section is adequate.

Deflection

Since the loading on this beam is not symmetrical, the calculations needed to determine the actual deflection are quite complex. A simpler approach is to calculate the deflection due to an equivalent UDL and compare it with the permitted limit of span/360. If this proves that the section is adequate then there would be no need to resort to more exact calculations.

The deflection should be based upon the unfactored imposed loads alone. These and the resulting shear and bending moment diagrams are shown in Figure 5.26. By equating the maximum bending moment of 129 kN m to the expression for the bending moment due to a UDL, an equivalent UDL can be calculated:

$$129 = \frac{WL}{8}$$

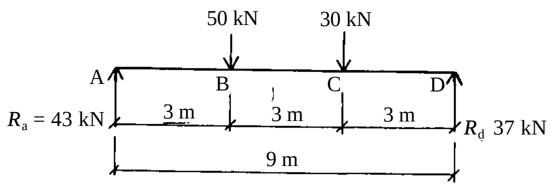
$$W = \frac{8 \times 129}{L} = \frac{8 \times 129}{9} = 115 \text{ kN}$$

This equivalent UDL of 115 kN may be substituted in the expression for the deflection of a simply supported beam:

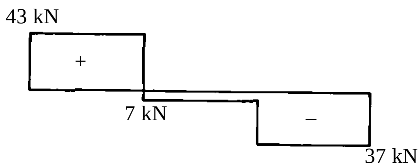
$$\text{Actual deflection } d_a = \frac{5}{384}\frac{WL^3}{EI} = \frac{5}{384} \times \frac{115 \times 10^3 \times 9000^3}{205 \times 10^3 \times 33\,400 \times 10^4} = 15.94 \text{ mm}$$

$$\text{Deflection limit } d_p = \frac{\text{span}}{360} = \frac{9000}{360} = 25 \text{ mm}$$

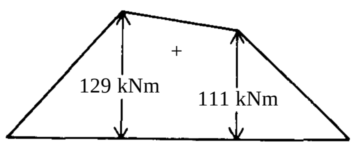
Thus $d_a < d_p$, and the beam is satisfactory in deflection.



(a) Unfactored imposed load diagram



(b) Shear force diagram



(c) Bending moment diagram

Figure 5.26 *Beam diagrams for unfactored imposed loads*

Web buckling and bearing

The web buckling and bearing requirements are not critical and therefore the calculations for these will be omitted.

Conclusion

That completes the check on the section, which has been shown to be adequate in bending, shear and deflection. Thus:

Adopt 457 × 191 × 74 kg/m UB.

5.11 Fabricated beams

In situations where standard rolled sections are found to be inadequate, consideration should be given to the following fabricated alternatives.

Compound beams

The strength of standard rolled sections can be increased by the addition of reinforcing plates welded to the flanges. Beams strengthened in this way are called compound beams. Examples are shown in Figure 5.27.

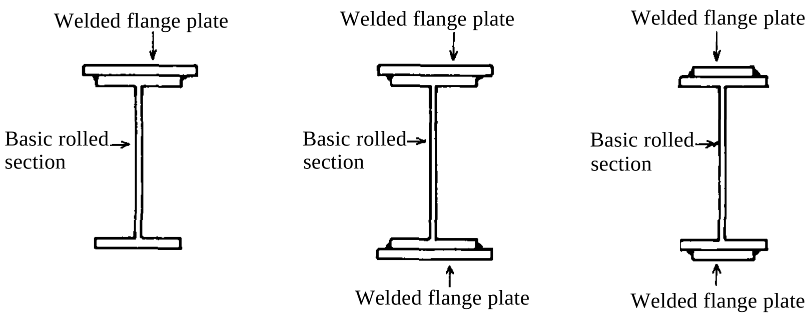


Figure 5.27 *Examples of compound beams*

Castellated beams

Standard rolled sections can be converted by cutting and welding into much deeper sections known as castellated beams. They offer a relatively simple method of increasing the strength of a section without increasing its weight.

To form a castellated beam, the basic rolled section is first flame cut along its web to a prescribed profile as shown in Figure 5.28a. Then the resulting two halves are rejoined by welding to form the castellated beam shown in Figure 5.28 b. The finished section is stronger in bending than the original but the shear strength is less. However, this usually only affects heavily loaded short span beams, and may be overcome where necessary by welding fitted plates into the end castellations as shown in Figure 5.28c.

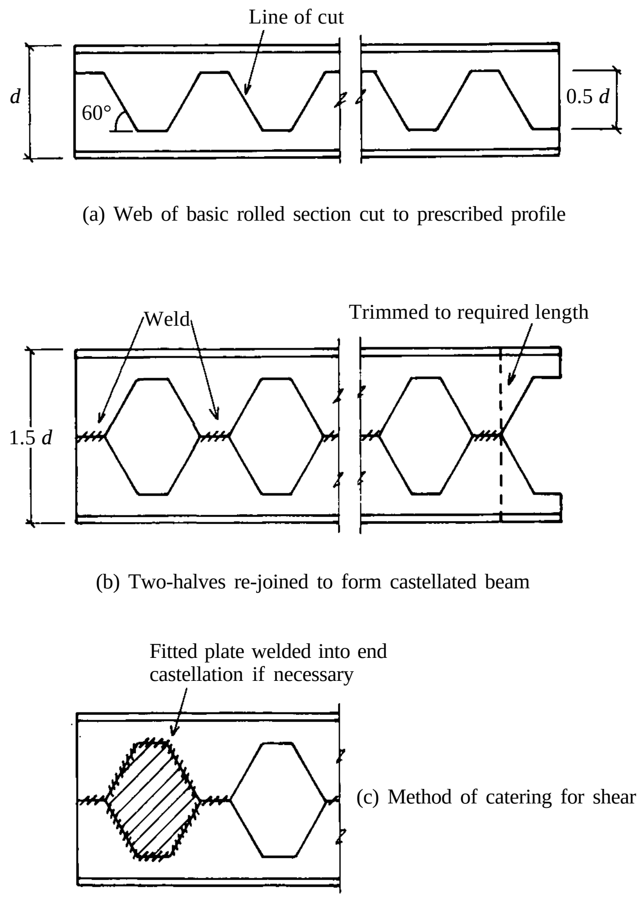


Figure 5.28 *Castellated beams*

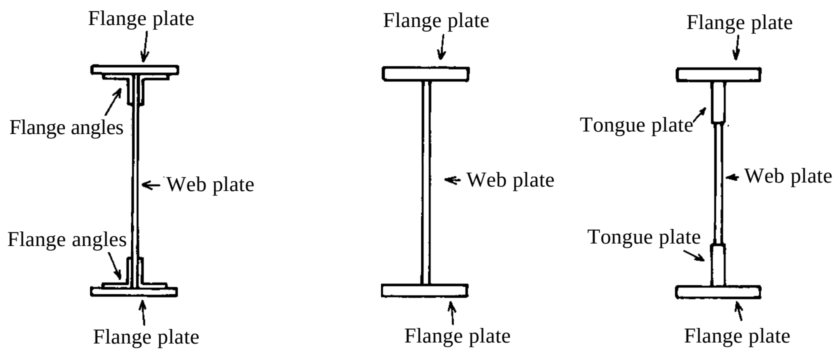


Figure 5.29 *Examples of plate girders*

Plate girders

Plate girders are used occasionally in buildings where heavy loads or long spans dictate, but more often they are used for bridges. They are formed from steel plates, sometimes in conjunction with angles, which are welded or bolted together to form I-sections. Three of the most common forms are illustrated in Figure 5.29.

Whilst plate girders can theoretically be made to any size, their depth for practical reasons should usually be between span/8 and span/12.

Lattice girders

Lattice girders are a framework of individual members bolted or welded together to form an open web beam. Two types of lattice girder commonly encountered are illustrated in Figure 5.30; they are the N-girder and the Warren girder. In comparison with the structural behaviour of beams, the top and bottom booms of a lattice girder resist bending and the internal members resist shear.

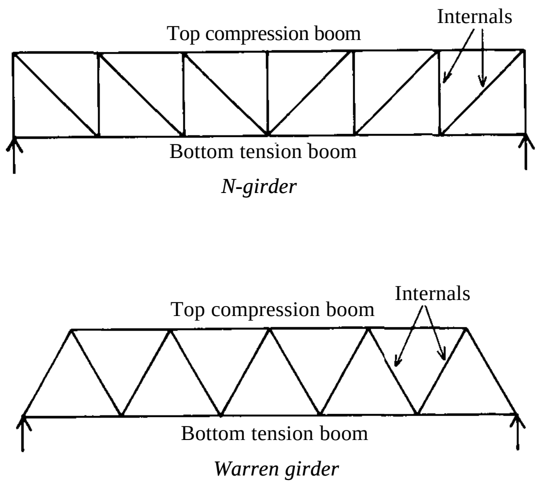


Figure 5.30 *Examples of lattice girders*

Generally their economical depth is between span/10 and span/15. Exceptions are short span heavily loaded girders, for which the depth may equal span/6, and long span lightly loaded roof girders, for which a depth of span/20 may suffice.

5.12 Columns

A steel column may be subject to direct compression alone, where the load is applied axially, or subject to a combination of compressive loading and bending due to the load being applied eccentrically to the member axes. It may also be subject to horizontal bending induced by lateral wind loading. However, the effect of wind loading on individual structural elements is not being considered in this manual.

Guidance for the design of axially loaded columns and axially loaded columns with moments is given in BS 5950 Part 1. The procedure for dealing with columns subject to axial load alone is first explained. This is then extended to include the interaction between compression and bending. Separate guidance is also given for the design of concrete cased columns and baseplates for columns.

The design of steel columns in this manual will therefore be considered under the following headings:

(a) Axially loaded columns
(b) Axially loaded columns with moments
(c) Cased columns
(d) Column base plates.

5.12.1 Axially loaded columns

A column supporting an axial load is subjected to direct compression. The compression resistance P_c of a column is given by the following expression:

$$P_c = A_g p_c$$

where A_g is the gross sectional area and p_c is the compressive strength. To ensure that a particular steel column is adequate, its compression resistance must be equal to or greater than the ultimate axial load F:

$$P_c \geq F$$

A steel column, because of its slender nature, will tend to buckle laterally under the influence of the applied compression. Therefore the compressive strength p_c is reduced to take account of the slenderness of the column. The slenderness l of an axially loaded column is given by the following expression:

$$l = \frac{L_E}{r}$$

where L_E is the effective length of the column and r is the radius of gyration of the section about the relevant axis, found from section tables.

The maximum slenderness of steel columns carrying dead and imposed loads is limited to 180. Values greater than this limit indicate that a larger section size is required.

Guidance on the nominal effective lengths to be adopted, taking end restraint into consideration, is given in BS 5950 Table 24. Additional guidance in relation to columns in certain single storey buildings and those forming part of a rigid frame are given in Appendices D and E respectively of the standard. The main effective length requirements for single storey steel sections are summarized here in Table 5.10.

Table 5.10 Effective length of steel columns

End condition	Effective length L_E
Restrained at both ends in position and direction	$0.7\,L$
Restrained at both ends in position and one end in direction	$0.85\,L$
Restrained at both ends in position but not in direction	$1.0\,L$
Restrained at one end in position and in direction and at the other end in direction but not in position	$1.5\,L$
Restrained at one end in position and in direction and free at the other end	$2.0\,L$

The compressive strength p_c depends on the slenderness l and the design strength of the steel p_y, or on a reduced design strength if the section is classified as slender. It was mentioned previously with respect to beams that the web and flanges of steel sections are comparatively slender in relation to their depth and breadth. Consequently the compressive force acting on a column could also cause local buckling of the web or flange before the full plastic stress is developed. This situation is avoided by reducing the stress capacity of the columns in relation to its section classification.

The column designs contained in this manual will be related to the use of UC sections, which are defined as H-sections in BS 5950. Since it can be shown that all UC sections are classified as at least semi-compact when used as axially loaded columns, no reduction in the design strength p_y because of local buckling will be necessary.

The value of the compressive strength p_c in relation to the slenderness l and the design strength p_y is obtained from strut tables given in BS 5950 as Table 27a–d. The specific table to use is indicated in Table 25 of the standard relative to the type of section employed. With respect to UC sections, the particular strut table to use is given here as Table 5.11.

Table 5.11 Selection of BS 5950 strut table

Section type	Thickness	Buckling axis x–x	Buckling axis y–y
Rolled H-section	Up to 40 mm	Table 27b	Table 27c
	Over 40 mm	Table 27c	Table 27d

5.12.2 Design summary for axially loaded steel columns

The general procedure for the design of axially loaded columns, using grade 43 UC sections, may be summarized as follows:

(a) Calculate the ultimate axial load F applied to the column.
(b) Determine the effective length L_E from the guidance given in Table 5.10.
(c) Select a trial section.
(d) Calculate the slenderness l from L_E/r and ensure that it is not greater than 180.
(e) Using the slenderness l and steel design strength p_y, obtain the compression strength p_c of the column from Table 27a–d of BS 5950.
(f) Calculate the compression resistance P_c of the column from the expression $P_c = A_g p_c$, where A_g is the gross sectional area of the column.
(g) Finally, check that the compression resistance P_c is equal to or greater than the ultimate axial load F.

Example 5.11

Design a suitable grade 43 UC column to support the ultimate axial load shown in Figure 5.31. The column is restrained in position at both ends but not restrained in direction.

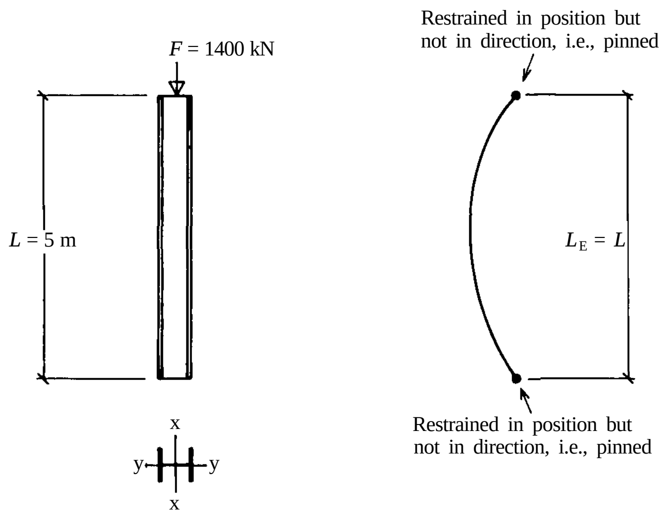


Figure 5.31 *Column load and effective lengths*

Ultimate axial load $F = 1400$ kN
Effective length $L_E = 1.0\,L = 5000$ mm

It is first necessary to assume a trial section for checking: try 203 × 203 × 86 kg/m UC. The relevant properties from section tables are as follows:

Flange thickness $T = 20.5$ mm
Area $A_g = 110$ cm$^2 = 110 \times 10^2$ mm^2
Radius of gyration $r_x = 9.27$ cm $= 92.7$ mm
Radius of gyration $r_y = 5.32$ cm $= 53.2$ mm

It has already been stated that all UC sections when acting as columns are classified as semi-compact; therefore it is unnecessary to show that the section is not slender.

The ultimate design strength p_y for grade 43 steel sections, from Table 5.1, is 275 N/mm^2 provided that the flange thickness does not exceed 16 mm. If the flange thickness is greater than 16 mm then p_y reduces to 265 N/mm^2. In this case $T =$ 20.5 mm > 16 mm, and therefore $p_y = 265$ N/mm^2.

The slenderness values are given by

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{5000}{92.7} = 54 < 180$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{5000}{53.2} = 94 < 180$$

These are satisfactory.

The relevant BS 5950 strut table to use may be determined from Table 5.11. For buckling about the *x–x* axis use Table 27b; for buckling about the *y–y* axis use Table 27c. Hence

For $\lambda_x = 54$ and $p_y = 265$ N/mm^2: $p_c = 223$ N/mm^2
For $\lambda_y = 94$ and $p_y = 265$ N/mm^2: $p_c = 133$ N/mm^2

Therefore p_c for design is 133 N/mm^2.

The compression resistance is given by

$$P_c = A_g p_c = 110 \times 10^2 \times 133 = 1\,463\,000 \text{ N} = 1463 \text{ kN} > 1400 \text{ kN}$$

That is, $P_c > F$. Thus:

Adopt 203 × 203 × 86 kg/m UC.

The *Steelwork Design Guide to BS 5950* produced by the Steel Construction Institute contains tables giving resistances and capacities for grade 43 UCs subject to both axial load and bending. A typical example for a number of UC sections is reproduced here as Table 5.12. From the table it may be seen that for the 203 × 203 × 86 kg/m UC section that has just been checked, the relevant axial capacity P_{cy} is given as 1460 kN. This again shows the advantage of such tables for reducing the amount of calculation needed to verify a section.

Example 5.12

If a tie beam were to be introduced at the mid-height of the column in Example 5.11, as shown in Figure 5.32, determine a suitable grade 43 UC section.

Ultimate axial load $F = 1400$ kN

By introducing a tie at mid-height on either side of the *y–y* axis, the section is effectively pinned at mid-height and hence the effective height about the *y–y* axis

Table 5.12 Universal columns subject to axial load and bending, steel grade 43: compression resistance P_{cx}, P_{cy} (kN) and buckling resistance moment M_b (kN m) for effective length L_e (m), and reduced moment capacity M_{rx}, M_{ry} (kN m) for ratios of axial load to axial load capacity F/P_z (abstracted from the *Steelwork Design Guide to BS 5950 Part 1*, published by the Steel Construction Institute)

Designation and capacities	L_e(m) F/P_z	1.5 0.05	2.0 0.10	2.5 0.15	3.0 0.20	3.5 0.25	4.0 0.30	5.0 0.35	6.0 0.40	7.0 0.45	8.0 0.50	9.0 0.55	10.0 0.60	11.0 0.65
203 × 203 × 86	P_{cx}	2920	2870	2810	2750	2680	2610	2450	2260	2040	1810	1580	1370	1190
P_z = 2920	P_{cy}	2740	2570	2400	2220	2030	1830	1460	1150	916	740	607	0	0
M_{cx} = 259	M_b	259	259	259	253	244	235	220	206	194	182	172	163	155
M_{cy} = 95	M_{bs}	259	259	259	259	259	254	233	211	190	168	148	130	115
p_yZ_y = 79	M_{rx}	258	253	245	235	221	207	194	180	166	152	138	123	108
	M_{ry}	95	95	95	95	95	95	95	95	95	95	95	90	82
203 × 203 × 71	P_{cx}	2410	2380	2330	2270	2220	2160	2020	1860	1680	1480	1290	1120	970
P_z = 2410	P_{cy}	2260	2130	1980	1830	1670	1510	1200	943	750	605	496	0	0
M_{cx} = 213	M_b	213	213	213	203	195	187	173	160	149	139	131	123	116
M_{cy} = 78	M_{bs}	213	213	213	213	213	207	190	172	154	137	120	106	93
p_yZ_y = 65	M_{rx}	211	207	200	191	181	170	158	147	135	124	112	100	88
	M_{ry}	78	78	78	78	78	78	78	78	78	78	78	73	67
203 × 203 × 60	P_{cx}	2080	2040	2000	1950	1900	1850	1730	1580	1400	1230	1060	914	789
P_z = 2080	P_{cy}	1940	1820	1700	1560	1410	1270	995	778	615	494	404	0	0
M_{cx} = 179	M_b	179	179	176	168	160	152	138	126	116	107	99	92	86
M_{cy} = 65	M_{bs}	179	179	179	179	179	173	158	143	127	112	98	85	75
p_yZ_y = 54	M_{rx}	178	175	170	162	153	144	134	124	114	105	94	84	74
	M_{ry}	65	65	65	65	65	65	65	65	65	65	65	62	57
203 × 203 × 52	P_{cx}	1830	1790	1750	1710	1670	1620	1510	1370	1220	1070	921	792	683
P_z = 1830	P_{cy}	1700	1600	1480	1360	1230	1100	865	676	534	429	351	0	0
M_{cx} = 156	M_b	156	156	152	144	137	130	117	106	96	87	80	74	69
M_{cy} = 57	M_{bs}	156	156	156	156	156	150	137	124	110	96	84	74	64
p_yZ_y = 47	M_{rx}	155	152	148	141	133	125	116	108	99	90	81	73	64
	M_{ry}	57	57	57	57	57	57	57	57	57	57	57	54	49
203 × 203 × 46	P_{cx}	1620	1580	1550	1510	1470	1430	1330	1210	1070	934	804	691	595
P_z = 1620	P_{cy}	1500	1410	1310	1200	1080	968	757	590	465	374	305	0	0
M_{cx} = 137	M_b	137	137	132	125	118	111	99	88	79	72	66	60	56
M_{cy} = 49	M_{bs}	137	137	137	137	137	131	120	108	95	83	73	64	55
p_yZ_y = 41	M_{rx}	136	133	129	124	116	109	102	94	86	79	71	63	55
	M_{ry}	49	49	49	49	49	49	49	49	49	49	49	47	43
152 × 152 × 37	P_{cx}	1280	1240	1210	1160	1110	1060	928	783	647	532	441	369	312
P_z = 1300	P_{cy}	1140	1030	910	787	671	568	411	306	0	0	0	0	0
M_{cx} = 85	M_b	85	82	77	72	68	64	57	52	47	43	39	36	34
M_{cy} = 30	M_{bs}	85	85	85	82	77	72	62	52	44	37	31	26	22
p_yZ_y = 25	M_{rx}	84	83	81	77	73	68	64	59	54	50	45	40	35
	M_{ry}	30	30	30	30	30	30	30	30	30	30	30	29	26
152 × 152 × 30	P_{cx}	1030	1000	969	933	893	847	740	622	512	420	347	290	246
P_z = 1050	P_{cy}	914	825	727	627	533	450	325	241	0	0	0	0	0
M_{cx} = 67	M_b	67	64	60	56	52	48	42	37	33	30	27	25	23
M_{cy} = 24	M_{bs}	67	67	67	65	61	57	49	41	34	28	24	20	17
p_yZ_y = 20	M_{rx}	67	66	64	61	58	54	51	47	43	39	35	32	28
	M_{ry}	24	24	24	24	24	24	24	24	24	24	24	23	21
152 × 152 × 23	P_{cx}	802	777	751	722	688	650	560	465	379	310	255	213	180
P_z = 820	P_{cy}	706	632	552	472	397	334	239	177	0	0	0	0	0
M_{cx} = 45	M_b	50	47	43	39	36	33	28	24	21	19	17	15	14
M_{cy} = 14	M_{bs}	50	50	50	47	44	41	35	29	24	20	17	14	12
p_yZ_y = 14	M_{rx}	50	49	48	46	44	41	38	35	33	30	27	24	21
	M_{ry}	17	17	17	17	17	17	17	17	17	17	17	17	16

F is factored axial load.

M_b is obtained using an equivalent slenderness = $nuvL_e/r$ with n = 1.0.

M_{bs} is obtained using an equivalent slenderness = 0.5 L/r.

Values have not been given for P_{cx} and P_{cy} if the values of slenderness are greater than 180.

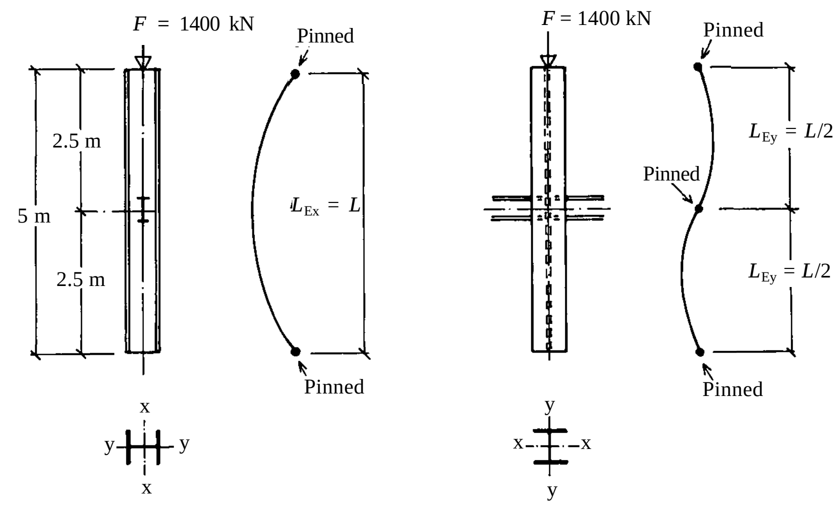


Figure 5.32 *Column load and effective lengths*

is halved. The effective height about the *x–x* axis will be unchanged. Thus

Effective length $L_{Ex} = 1.0L = 5000$ mm

Effective length $L_{Ey} = \frac{L}{2} = \frac{5000}{2} = 2500$ mm

It is again necessary to assume a trial section for checking: try 203 × 203 × 52 kg/m UC. The relevant properties from section tables are as follows:

Flange thickness $T = 12.5$ mm
Area $A_g = 66.4\ \text{cm}^2 = 66.4 \times 10^2\ \text{mm}^2$
Radius of gyration $r_x = 8.9$ cm = 89 mm
Radius of gyration $r_y = 5.16$ cm = 51.6 mm

Here $T = 12.5$ mm < 16 mm, and therefore $p_y = 275\ \text{N/mm}^2$. The slenderness values are given by

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{5000}{89} = 56 < 180$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{2500}{51.6} = 48 < 180$$

These are satisfactory.

The relevant strut tables to use, as determined from Table 5.11, are the same as in Example 5.11. Hence

For $\lambda_x = 56$ and $p_y = 275$ N/mm^2: $p_c = 227$ N/mm^2
For $\lambda_y = 48$ and $p_y = 275$ N/mm^2: $p_c = 224$ N/mm^2

It should be noted that even though the slenderness about the *x–x* axis is greater than that about the *y–y* axis, the lower value of p_c for λ_y will be the design criterion. Therefore p_c for design is 224 N/mm^2.

The compression resistance is given by

$$P_c = A_g p_c = 66.4 \times 10^2 \times 224 = 1\,487\,360 \text{ N} = 1487 \text{ kN} > 1400 \text{ kN}$$

That is, $P_c > F$. Thus:

Adopt 203 × 203 × 52 kg/m UC.

The value of P_{cy} given in Table 5.12 for this section is 1480 kN.

It should be appreciated that the purpose of this example is not to advocate the introduction of tie beams in order to reduce the effective length of columns, but to illustrate the advantage of taking such beams or similar members into account when they are already present.

Figure 5.33 illustrates the effect on the slenderness that a similar mid-height tie would have if the column had been restrained in position and direction at both the cap and the base. The design procedure for the column would then be exactly the same as this example.

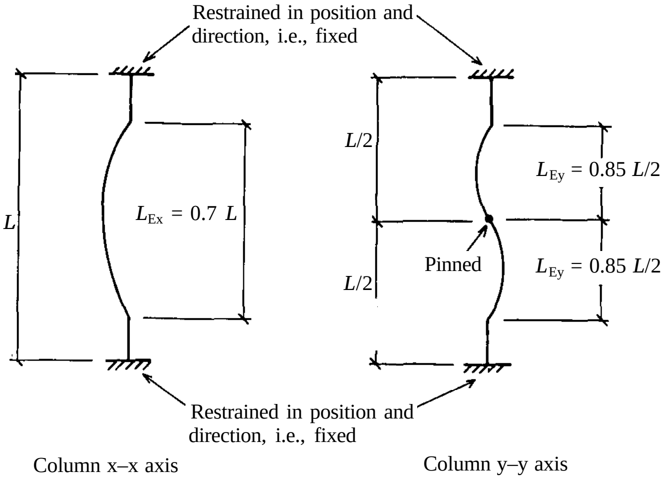


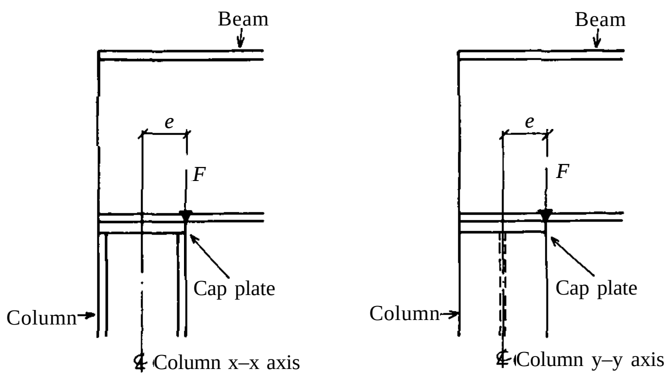
Figure 5.33 *Column effective lengths*

5.12.3 Axially loaded columns with nominal moments

The design of steel elements dealt with in this manual is based upon the principles of simple design. This assumes that the end connections between members are such that they cannot develop any significant restraint moments.

In practice the loads supported by columns are seldom applied concentrically, and therefore the effect of eccentric loading should be considered. For simple construction, where the end connections are not intended to transmit significant bending moments, the degree of eccentricity may be taken as follows:

(a) For a beam supported on a column cap plate, such as that shown in Figure 5.34a or b, the load is taken as acting at the face of the column.
(b) Where beams are connected by simple connections to the face of a column, as in Figure 5.35a and b, the load should be taken as acting at 100 mm from the column face, as shown in Figure 5.36.
(c) When a roof truss is supported on a column cap plate, as shown in Figure 5.37, and the connection cannot develop significant moments, the eccentricity may be neglected.



(a) Beam spanning at right angles to the x–x axis

(b) Beam spanning at right angles to the y–y axis

Figure 5.34 *Beams supported on a column cap plate*

A load applied eccentrically will induce a nominal bending moment in the column equal to the load times the eccentricity:

$$M_e = Fe$$

The effect on the column of this moment must be examined in conjunction with the axial load.

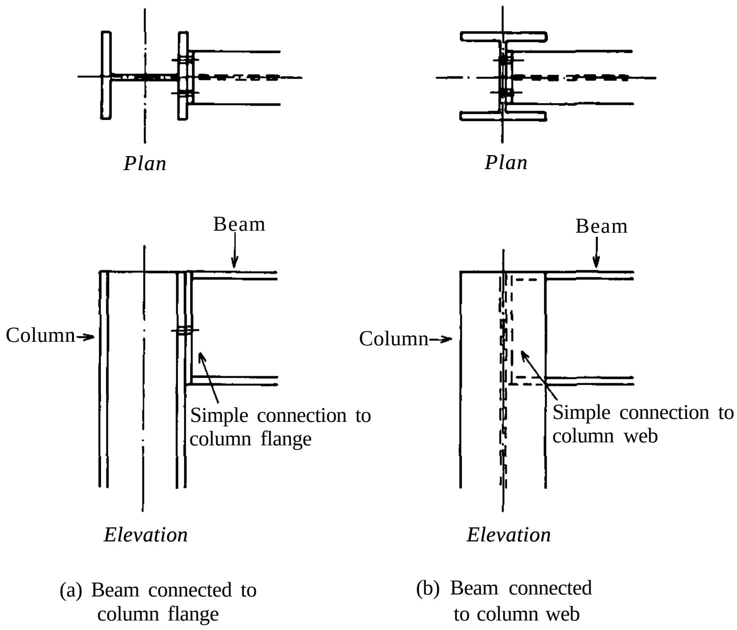


Figure 5.35 *Beams connected to the face of a column*

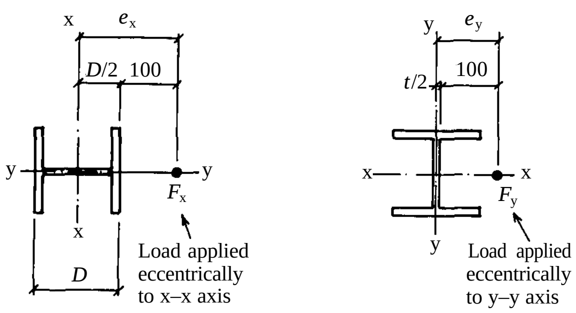


Figure 5.36 *Load eccentricity for beams connected to the face of a column*

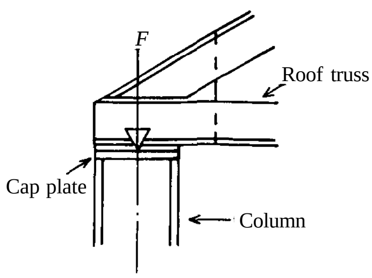


Figure 5.37 *Column supporting a roof truss, the load from which is transmitted concentrically*

Generally there are two separate checks that need to be applied to axially loaded columns with moments; they are a local capacity check and an overall buckling check.

Local capacity check

The local capacity of a column should be checked at the point of greatest bending moment and axial load. It will vary depending on the section classification, and therefore two relationships are given in BS 5950. One is for semi-compact and slender cross-sections, whilst the other is for plastic and compact cross-sections. For simplicity the relationship for semi-compact and slender cross-sections may be used to check all columns except those designed by plastic analysis. Therefore if elastic analysis is employed only one relationship need be satisfied, which is as follows:

$$\frac{F}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

F	applied axial load
A_g	gross sectional area, from section tables
p_y	design strength of the steel
M_x	applied moment about the major axis
M_{cx}	moment capacity about the major axis in the absence of axial load; see Section 5.10.2 for beams
M_y	applied moment about the minor axis
M_{cy}	moment capacity about the minor axis in the absence of axial load; see Section 5.10.2 for beams

It should be noted that if the column section is classified as slender, the design strength p_y of the steel would be reduced. This does not apply to any UC sections since none is classed as slender.

Overall buckling check

A simplified approach and a more exact approach are offered in BS 5950 for the overall buckling check. The simplified approach must always be used to check columns subject to nominal moments. Since only columns with nominal moments are dealt with in this manual only the simplified approach will be considered here, for which the following relationship must be satisfied:

$$\frac{F}{A_g p_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y Z_y} \leq 1$$

where

F	applied axial load
A_g	gross sectional area, from section tables
p_c	compressive strength
m	has value 1 when only nominal moments are applied
M_x	applied moment about the major axis
M_b	buckling resistance capacity about the major axis
M_y	applied moment about the minor axis
p_y	design strength of the steel
Z_y	elastic section modulus about the minor axis, from section tables

It should be noted that when $m = 1$ the overall buckling check will always control the design. Therefore for columns supporting only nominal moments it is not necessary to carry out the local capacity check discussed in the previous section.

The buckling resistance capacity M_b of the section about the major axis is obtained from the following expression:

$$M_b = p_b S_x$$

where p_b is the bending strength and S_x is the plastic modulus of the section about the major axis, obtained from section tables. The bending strength for columns is obtained from BS 5950 Table 11, reproduced earlier as Table 5.5. It depends on the steel design strength p_y and the equivalent slenderness l_{LT}, which for columns supporting only nominal moments may be taken as

$$l_{LT} = 0.5\frac{L}{r_y}$$

where L is the distance between levels at which both axes are restrained, and r_y is the radius of gyration of the section about its minor axis, from section tables.

5.12.4 Design summary for axially loaded steel columns with nominal moments

The procedure for the design of axially loaded columns with nominal moments, using grade 43 UC sections, may be summarized as follows:

(a) Calculate the ultimate axial load F applied to the column.

(b) Select a trial section.

(c) Calculate the nominal moments M_x and M_y about the respective axes of the column.

(d) Determine the overall effective length L_E from the guidance given in Table 5.10

(e) Calculate the slenderness l from L_E/r and ensure that it is not greater than 180.

(f) Using the slenderness l and the steel design strength p_y, obtain the compression strength p_c from Table 27a–d of BS 5950.

(g) Obtain the bending strength p_b from Table 5.5 using the steel design strength p_y and the equivalent slenderness l_{LT}, which may be taken as $0.5\,L/r_y$ for columns subject to nominal moments.

(h) Calculate M_b from the expression $M_b = p_b S_x$.

(i) Ensure that the following relationship is satisfied:

$$\frac{F}{A_g p_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y Z_y} \text{ £ } 1$$

Example 5.13

Design a suitable grade 43 UC column to support the ultimate loads shown in Figure 5.38. The column is effectively held in position at both ends and restrained in direction at the base but not at the cap.

Ultimate axial load $F = 125 + 125 + 285 + 5 = 540\text{ kN} = 540 \times 10^3\text{ N}$

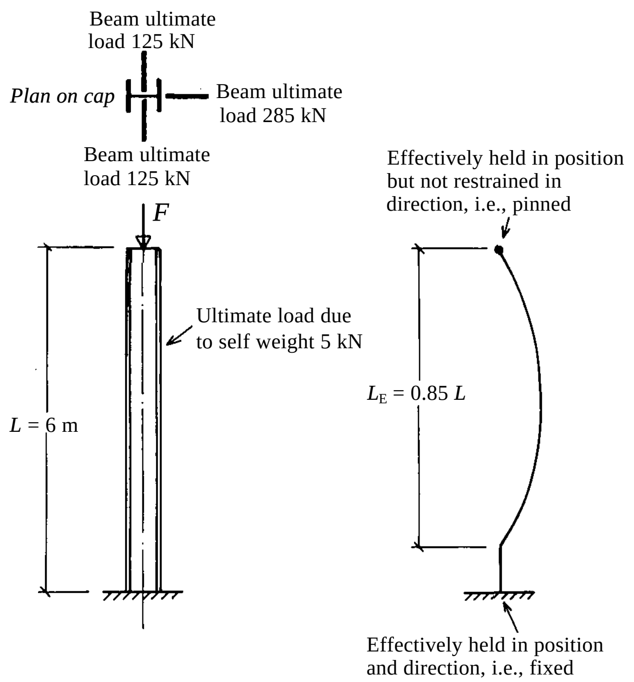


Figure 5.38 *Column loads and effective length*

Once again it is necessary to assume a trial section for checking: try $203 \times 203 \times 60$ kg/m UC. The relevant properties from section tables are as follows:

Depth D = 209.6 mm; width B = 205.2 mm
Flange thickness T = 14.2 mm
Web thickness t = 9.3 mm
Area $A_g = 75.8\ \text{cm}^2 = 75.8 \times 10^2\ \text{mm}^2$
Radius of gyration r_x = 8.96 cm = 89.6 mm
Radius of gyration r_y = 5.19 cm = 51.9 mm
Plastic modulus $S_x = 652\ \text{cm}^3 = 652 \times 10^3\ \text{mm}^3$

It has already been stated that all UC sections are semi-compact, and therefore it is unnecessary to show that the section is not slender.

The eccentricity e_x is given by

$$e_x = \frac{D}{2} + 100 = \frac{209.6}{2} + 100 = 204.8\ \text{mm}$$

Then

$$\begin{aligned} \text{Nominal moment } M_x &= \text{beam reaction} \times e_x \\ &= 285 \times 204.8 = 58\,368\ \text{kN mm} = 58.368 \times 10^6\ \text{N mm} \end{aligned}$$

Since the beam reactions are the same on either side of the y–y axis there will be no bending about this axis: therefore $M_y = 0$.

It is not necessary to check the local buckling capacity of columns subject to nominal moments. The overall buckling check using the simplified approach should be carried out to ensure that the following relationship is satisfied:

$$\frac{F}{A_g p_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y Z_y} \le 1$$

The compression strength p_c is calculated as follows. First, $T = 14.2\ \text{mm} < 16\ \text{mm}$. Therefore $p_y = 275\ \text{N/mm}^2$. The slenderness values are given by

$$\lambda_x = \frac{L_E}{r_x} = \frac{0.85\,L}{r_x} = \frac{0.85 \times 6000}{89.6} = 57 < 180$$

$$\lambda_y = \frac{L_E}{r_y} = \frac{0.85\,L}{r_y} = \frac{0.85 \times 6000}{51.9} = 98 < 180$$

These are satisfactory.

The relevant BS 5950 strut tables to use may be determined from Table 5.11. For buckling about the x–x axis use Table 27b; for buckling about the y–y axis use Table 27c. Hence

For $\lambda_x = 57$ and $p_y = 275\ \text{N/mm}^2$: $p_c = 225\ \text{N/mm}^2$
For $\lambda_y = 98$ and $p_y = 275\ \text{N/mm}^2$: $p_c = 129\ \text{N/mm}^2$

Therefore p_c for design is 129 N/mm^2.

For columns subject to nominal moments, m may be taken as 1.0.

The buckling resistance moment M_b for columns subject to nominal moments is calculated as follows. First,

$$\lambda_{LT} = \frac{0.5L}{r_y} = \frac{0.5 \times 6000}{51.9} = 58$$

Next, p_b = 218 N/mm^2 by interpolation from Table 5.5. Therefore

$$M_b = p_b S_x = 218 \times 652 \times 10^3 = 142.14 \times 10^6 \text{ N mm}$$

Hence

$$\frac{F}{A_g p_c} + \frac{mM_x}{M_b} + \frac{mM_y}{p_y Z_y} = \frac{540 \times 10^3}{75.8 \times 10^2 \times 129} + \frac{1 \times 58.368 \times 10^6}{142.14 \times 10^6} + 0$$
$$= 0.55 + 0.41 + 0 = 0.96 < 1.0$$

Adopt 203 × 203 × 60 kg/m UC.

5.12.5 Cased columns

If steel columns are to be cased in concrete, for fire protection perhaps, structural advantage may be taken of the casing if certain requirements are met with respect to the concrete and reinforcement. The requirements in relation to UC sections are basically as follows:

(a) The steel section is unpainted and free from oil, grease, dirt or loose rust and millscale.
(b) The steel section is solidly encased in ordinary dense structural concrete of at least grade 30 to BS 8110.
(c) The surface and edges of the flanges of the steel section have a concrete cover of not less than 50 mm.
(d) The casing is reinforced using steel fabric, reference D 98, complying with BS 4483. Alternatively steel reinforcement not less than 5 mm diameter, complying with BS 4449 or BS 4482, may be used in the form of a cage of longitudinal bars held by closed links at a maximum spacing of 200 mm. The maximum lap of the reinforcement and the details of the links should comply with BS 8110.
(e) The reinforcement is so arranged as to pass through the centre of the concrete cover.

A typical cross-section through a cased UC satisfying these requirements is shown in Figure 5.39.

The allowable load for concrete cased columns is based upon certain empirical rules given in BS 5950. Those relating to axially loaded cased columns are as follows:

(a) The effective length L_E is limited to the least of 40 b_c or 100 b_c^2/d_c or 250 r, where

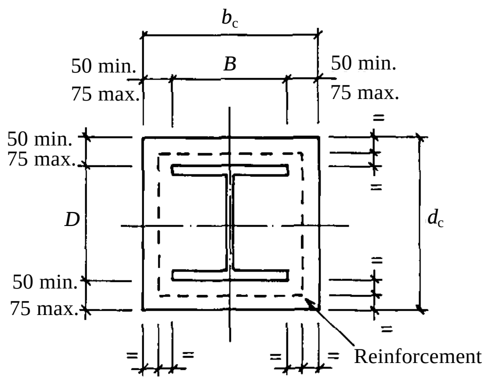


Figure 5.39 *Typical cross-section through a cased UC column*

b_c minimum width of solid casing within the depth of the steel section, as indicated in Figure 5.39

d_c minimum depth of solid casing within the width of the steel section, as indicated in Figure 5.39

r minimum radius of gyration of the uncased steel section, that is r_y for UC sections

(b) The radius of gyration r_y of the cased section should be taken as $0.2b_c$ but never more than $0.2(b + 150)$ mm. This implies that any casing above 75 mm cover should be ignored for structural purposes. The radius of gyration r_x should be taken as that of the uncased steel section.

(c) The compression resistance P_c of a cased column should be determined from the following expression:

$$P_c = \left(A_g + 0.45\frac{f_{cu}}{p_y}A_c\right)p_c$$

However, this should not be greater than the short strut capacity of the section, given by

$$P_{cs} = \left(A_g + 0.25\frac{f_{cu}}{p_y}A_c\right)p_y$$

where

A_c gross sectional area of the concrete, ($b_c d_c$ in Figure 5.39) but neglecting any casing in excess of 75 mm or any applied finish

A_g gross sectional area of the steel section

f_{cu} characteristic concrete cube strength at 28 days, which should not be greater than 40 N/mm^2

p_c compressive strength of the steel section determined in the manner described for uncased columns in Section 5.12.1, but using the r_y and r_x of the cased section

p_y design strength of the steel: $p_y \leq 355$ N/mm^2

P_{cs} short strut capacity, that is the compression resistance of a cased strut of zero slenderness

When a cased column is subject to axial load and bending it must satisfy the following relationships:

(a) Local capacity check:

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

(b) Overall buckling resistance:

$$\frac{F_c}{P_c} + \frac{mM_x}{M_b} + \frac{mM_y}{M_{cy}} \leq 1$$

The radius of gyration r_y for calculating the buckling resistance moment M_b of a cased column should be taken as the greater of the r_y of the uncased section or $0.2(B + 100)$ mm, where B is as indicated in Figure 5.39. The value of M_b for the cased section must not exceed 1.5 M_b for the same section uncased.

Example 5.14

Determine the compression resistance of the grade 43 203 × 203 × 86 kg/m UC column shown in Figure 5.40, which is structurally cased to the minimum requirements of BS 5950. The column is effectively held in position at both ends but not restrained in direction, as indicated in Figure 5.41

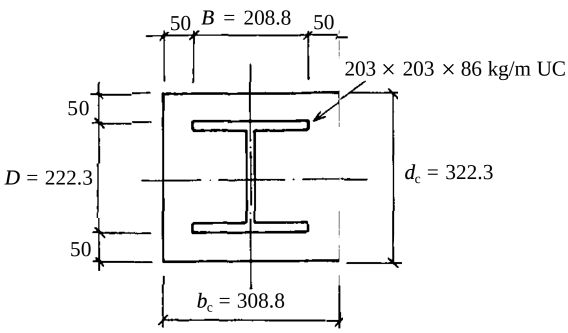


Figure 5.40 *Cross-section through cased column*

The properties of the cased section are as follows, from section tables where appropriate:

Gross area of concrete $A_c = b_c d_c = 308.8 \times 322.3 = 99\ 526$ mm

Gross sectional area of steel section $A_g = 110$ cm^2 $= 110 \times 10^2$ mm^2

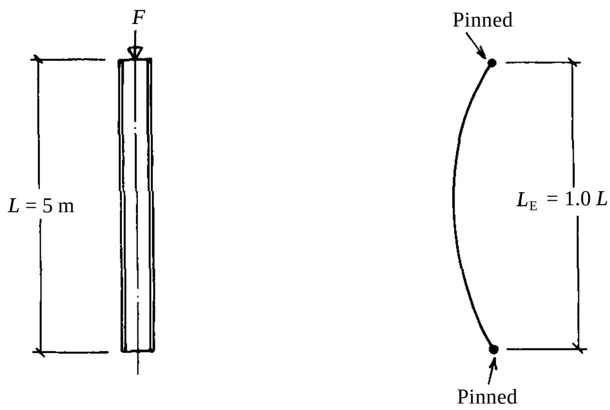


Figure 5.41 *Effective length of column*

$r_x = r_x$ of uncased section = 9.27 cm = 92.7 mm
r_y of uncased section = 5.32 cm = 53.2 mm
r_y of cased section = $0.2b_c = 0.2 \times 308.8 = 61.8$ mm
Effective length $L_E = L = 5000$ mm

Check that the effective length does not exceed the limiting values for a cased column:

$$40b_c = 40 \times 308.8 = 12\ 352 \text{ mm} > 5000 \text{ mm}$$

$$\frac{100b_c^2}{d_c} = \frac{100 \times 308.8^2}{322.3} = 29\,587 \text{ mm} > 5000 \text{ mm}$$

$$250r_y \text{ of uncased section} = 250 \times 53.2 = 13\ 300 > 5000 \text{ mm}$$

Here $T = 20.5$ mm > 16 mm. Therefore $p_y = 265$ N/mm^2. The slenderness values are given by

$$l_x = \frac{L_E}{r_x} = \frac{5000}{92.7} = 54 < 180$$

$$l_y = \frac{L_E}{r_y} = \frac{5000}{61.8} = 81 < 180$$

These are satisfactory.

The relevant BS 5950 strut tables to use may be determined from Table 5.11. For buckling about the *x–x* axis use Table 27b; for buckling about the *y–y* axis use Table 27c. Hence

For $l_x = 54$ and $p_y = 265$ N/mm^2: $p_c = 223$ N/mm^2
For $l_y = 81$ and $p_y = 265$ N/mm^2: $p_c = 155$ N/mm^2

Therefore p_c for design is 155 N/mm^2.

The compression resistance is given by

$$P_c = \left(A_g + 0.45 \frac{f_{cu}}{P_y} A_c\right) p_c$$

$$= \left(110 \times 10^2 + 0.45 \times \frac{20}{265} \times 99\ 526\right) 155$$

$$= (11\ 000 + 3380)155 = 14\ 380 \times 155 = 2\ 228\ 900 \text{ N} = 2229 \text{ kN}$$

This must not be greater than the short strut capacity P_{cs} of the section, given by

$$P_{cs} = \left(A_g + 0.25 \frac{f_{cu}}{p_y} A_c\right) p_y$$

$$= \left(11\ 000 + 0.25 \times \frac{20}{265} \times 99\ 526\right) 265$$

$$= (11\ 000 + 1878)265 = 12\,878 \times 265 = 3\,412\ 670 \text{ N} = 3413 \text{ kN} > 2229 \text{ kN}$$

Therefore the compression resistance of the cased column is 2229 kN. This may be compared with the compression resistance of 1463 kN for the same section uncased that was calculated in Example 5.11. Thus the load capacity of the section when cased has increased by 52 per cent.

5.12.6 Column baseplates

The column designs contained in this manual relate to axially loaded columns and columns subject to nominal moments at the cap. Therefore only the design of baseplates subject to compressive loading will be included here.

Empirical rules are given in BS 5950 for the design of slab baseplates, as illustrated in Figure 5.42, when subject to compressive loads only. When a column is concentrically loaded it may be assumed that the load at the base is transmitted uniformly over the area of the steel baseplate to the foundation concrete.

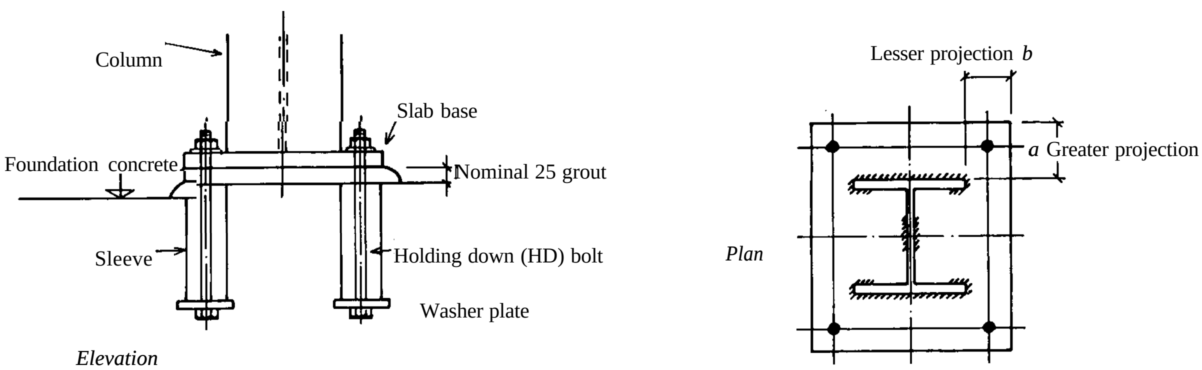


Figure 5.42 *Typical slab base*

The bearing strength for concrete foundations may be taken as $0.4f_{cu}$, where f_{cu} is the characteristic concrete cube strength at 28 days as indicated in Table 5.13. This enables the area of baseplate to be calculated and suitable plan dimensions to be determined. The baseplate thickness is then determined from the following expression:

$$t = \left[\frac{2.5}{p_{yp}} w\,(a^2 - 0.3\,b^2)\right]^{1/2}$$

but t must not be less than the flange thickness of the column. In this expression,

- a greater projection of the plate beyond the column (see Figure 5.42)
- b lesser projection of the plate beyond the column (see Figure 5.42)
- w pressure on the underside of the plate assuming a uniform distribution (N/mm^2)
- p_{yp} design strength of the plate, which may be taken as p_y given in Table 5.1, but not greater than 270 N/mm^2

Table 5.13 Bearing strength for concrete foundations

Concrete grade	Characteristic cube strength at 28 days f_{cu} (N/mm^2)	Bearing strength $0.4\,f_{cu}$ (N/mm^2)
C30	30	12.0
C35	35	14.0
C40	40	16.0
C45	45	18.0
C50	50	20.0

Example 5.15

Design a suitable slab baseplate for a 203 × 203 × 86 kg/m UC supporting an ultimate axial load of 1400 kN if the foundation is formed from grade 30 concrete. It should be noted that this is the column for which the steel section was originally designed in Example 5.11.

Grade 30 concrete f_{cu} = 30 N/mm^2

Bearing strength from Table 5.13 = 12 N/mm^2

$$\text{Area of slab baseplate required} = \frac{\text{axial load}}{\text{bearing strength}} = \frac{1400 \times 10^3}{12} = 116\,667\ \text{mm}^2$$

Since the column section is basically square, provide a square baseplate. The baseplate side = $\sqrt{(116\,667)}$ = 342 mm. For practical reasons use a 350 mm square baseplate, for which the plan configuration taking into account the actual dimensions of the UC will be as indicated in Figure 5.43.

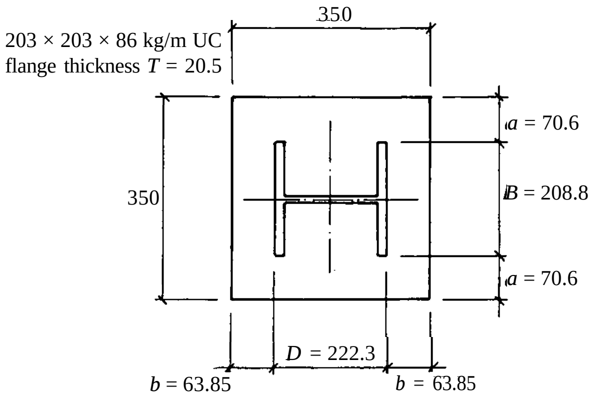


Figure 5.43 *Plan on baseplate*

The baseplate thickness is determined using the BS 5950 empirical expression:

$$t = \left[\frac{2.5}{p_{yp}} w(a^2 - 0.3\,b^2)\right]^{1/2}$$

where a and b are the dimensions shown in Figure 5.43. The bearing pressure is given by

$$w = \frac{1400 \times 10^3}{350 \times 350} = 11.43 \text{ N/mm}^2$$

The design strength of the plate for grade 43 steel is obtained from Table 5.1, but must not be greater than 270 N/mm^2. Since the flange thickness T of the UC column is 20.5 mm and the baseplate thickness t must not be less than this, the design strength from Table 5.1 will be 265 N/mm^2. Therefore

$$t = \left[\frac{2.5}{265} \times 11.43(70.6^2 - 0.3 \times 63.85^2)\right]^{1/2}$$

$$= 20.14 \text{ mm} < 20.5 \text{ mm UC flange thickness}$$

Use a 25 mm thick baseplate. Thus finally:

Adopt a 350 × 350 × 25 baseplate.

5.13 Connections

The design of connections usually follows the design of the principal components of a steel framed structure, and is normally regarded as part of the detailing process.

Connections may be bolted, welded or a combination of both. They must be proportioned with proper regard to the design method adopted for the structure as a whole. Therefore the bolts or welds making up a connection must be capable of transmitting all direct forces and resisting any bending moments.

The design of bolted or welded connections is beyond the scope of this manual, which is concerned with the design of individual elements. The British Constructional Steelwork Association publishes a book on the design of connections for joints in simple construction which would be useful for anyone with a particular interest in this topic. This and other sources of information relating to steel design are listed in the reference section.

5.14 References

BS 4 Structural steel sections.
Part 1 1980 Specification for hot-rolled sections.

BS 4360 1990 British Standard Specification for weldable structural steels.

BS 4848 Specification for hot-rolled structural steel sections.
Part 2 1991 Hollow sections.
Part 4 1972 Equal and unequal angles.

BS 5493 1977 Code of practice for protective coating of iron and steel structures against corrosion.

BS 5950 Structural use of steelwork in building.
Part 1 1990 Code of practice for design in simple and continuous construction: hot rolled sections.
Part 2 1985 Specification for materials, fabrication and erection: hot rolled sections.

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