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.

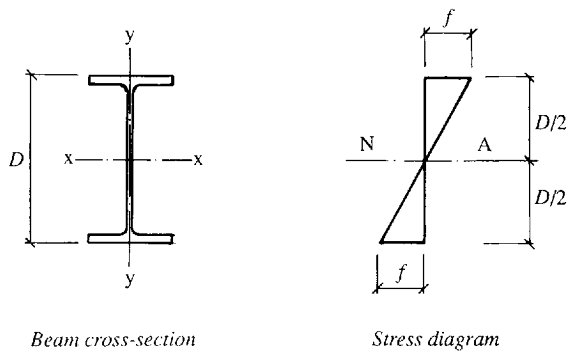

Figure 5.6 *Elastic stress distribution*

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:

$$\mathrm{MR} \geqslant \mathrm{BM}$$

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

$$M_{\mathrm{cx}} = p_{\mathrm{y}} S_{\mathrm{x}}$$

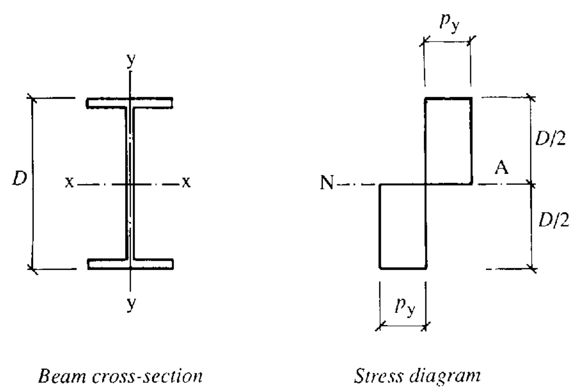

Figure 5.7 *Plastic stress distribution*

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

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

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:

$$M_{cx} \geqslant M_u$$

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	8.5ε	9.5ε	15ε
d/t	79ε	98ε	120ε

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