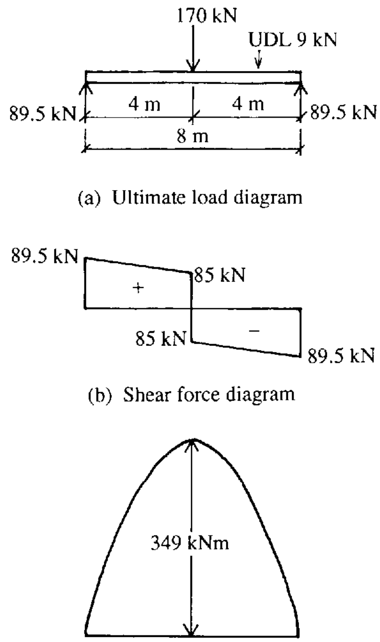

(c) Bending moment diagram

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 $F_v = 85\,\text{kN}$; $M = 349\,\text{kN m}$

Shear capacity of section $P_v = 0.6 p_y t D$

$$= 0.6 \times 265 \times 9.9 \times 461.3 = 726\,132\,\text{N}$$

$$= 726\,\text{kN} > 85\,\text{kN}$$

Furthermore $0.6P_v = 0.6 \times 726 = 435.6\,\text{kN}$. 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 $F_v = 89.5\,\text{kN}$ $M = 0$

Shear capacity of section $P_v = 726\,\text{kN} > 89.5\,\text{kN}$

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,

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

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,

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

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

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,

$$\delta_a \leqslant \delta_p$$

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.

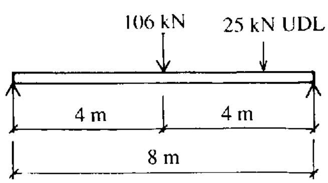

Figure 5.19 *Unfactored imposed loads*

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 cm⁴. The deflection limit is given by

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

The actual deflection is

$$\delta_a = \frac{5}{384}\frac{WL^3}{EI} + \frac{1}{48}\frac{WL^3}{EI}$$

$$= \frac{5}{384} \times \frac{25 \times 10^3 \times 8000^3}{205 \times 10^3 \times 37\,100 \times 10^4}$$

$$+ \frac{1}{48} \times \frac{106 \times 10^3 \times 8000^3}{205 \times 10^3 \times 37\,100 \times 10^4}$$

$$= 2.19 + 14.87 = 17.06\,\text{mm} < 22.22\,\text{mm}$$

That is $\delta_a < \delta_p$, and therefore the section is adequate in deflection.