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

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

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 β, 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.

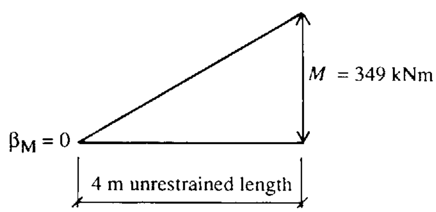

Figure 5.12 *Equivalent bending moment diagram for the unrestrained length*

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

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

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 λ_{LT}. We have $p_y = 265\ \text{N/mm}^2$ and

$$\lambda_{LT} = nuv\lambda$$

where $n = 1.0$, $u = 0.87$ from section tables, and $\lambda = L_E/r_y$. In this instance $L_E = 1.0L$ 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

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

Now $x = 30$ from section tables. Hence $\lambda/x = 122.7/30 = 4.09$. 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 $p_b = 138.24\,\text{N/mm}^2$ by interpolation from Table 5.5. Thus finally

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

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:

$M_{cx} = 429\,\text{kN m}$; and $M_b = 223\,\text{kN m}$ 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?

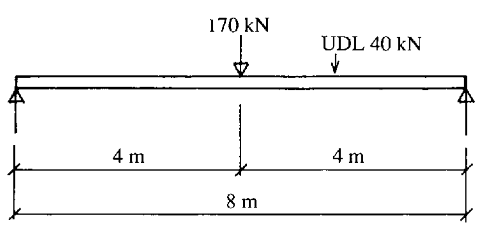

Figure 5.13 *Ultimate load diagram*

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 ($S_x = 1830\,\text{cm}^3$). 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.