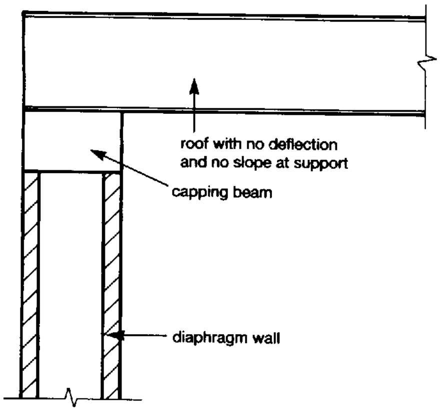

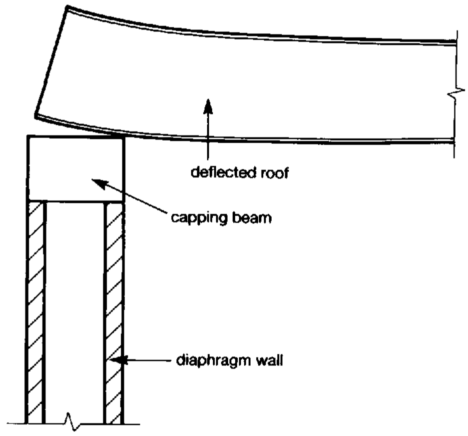

Figure 32: Eccentricity of roof loading

Such an extreme case is hardly likely to occur in practice, since there will be some dispersal of the contact pressure through the capping beam. However, this dispersion is unlikely to be sufficient to cause the outer and inner leaves to be loaded equally. The inner leaf will be more heavily stressed than the outer leaf. For bonded cross-ribs this could be considered as a local bearing stress, since the roof beam loads are applied at intervals. Clause 34 of BS 5628 : Part 1 allows up to 50% increase in the local stress. The rib also probably disperses the excess stress from the inner to the outer leaf. For tied cross-ribs, the designer should assess the effect of the eccentricity on an individual job basis, taking account of the magnitude of the vertical loading and the contribution of the shear ties in transferring the load from inner to outer leaves.

The problem of eccentricity can often be more economically controlled by detailing the bearing of the roof beams on the capping beams so that the load is applied where the designer wants it (Figure 33).

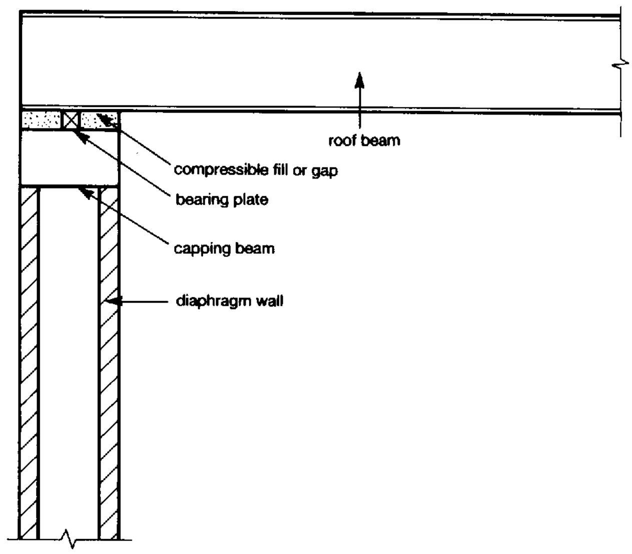

Figure 33: Method of reducing roof eccentricity

2.5.5 Capacity reduction factor β

The capacity reduction factor, which allows for the effects of slenderness and eccentricity of loading in determining the design vertical load resistance of the wall, may now be obtained from Table 7 of BS 5628 : Part 1 using the actual thickness as the effective thickness.

3 Design procedure and worked examples

A suggested procedure for the design of a diaphragm wall in an open-plan single-storey building is as follows:

Stage 1 Calculate the characteristic and design loads from dead, superimposed and wind loading on the structure.

Stage 2 Select a trial section using Figures 28 and 29.

Stage 3 Calculate size of ring beam if required.

Stage 4 Assume a block strength (or brick strength) and mortar designation.

Stage 5 Check the external leaf spanning between cross-ribs for adequate thickness.

Stage 6 Calculate the design wind moment and the stability moment of resistance at the base of the wall.

Stage 7 Calculate the design flexural stresses at the location of maximum applied moment in the wall, M_w.

Stage 8 Calculate the flexural resistance of the masonry at the level of maximum M_w and compare it with the design stresses calculated in stage 7.

Stage 9 Calculate the shear stress in the cross-ribs.

Stage 10 Design the shear resistance of the cross-ribs.

Stage 11 Summarize conditions (a) (b) (c) and (d) for cross-rib spacing (see Section 2.2.1).

Stage 12 Design the roof plate and transverse shear walls (these aspects of the design are outside the scope of this design guide).

Stage 13 Check the loading combinations of (a) dead plus imposed plus wind and (b) dead plus imposed only.

Worked example 1: Warehouse building

The warehouse measures 40 x 75 x 7.5 m high and is shown in Figure 34. It is to be designed in concrete blockwork, using diaphragm wall construction for its main vertical structure. There are no internal walls to provide intermediate support to the external walls. Normal testing of materials and minimum supervision of the workmanship will be employed during the building programme.

Solid concrete facing blocks with a compressive strength of 7.0 N/mm^2 will be used throughout the building, and are assumed to have a density of 2000 kg/m^3 (20 kN/m^3).

Stage 1. Loading

(a) Wind loads

The basic wind pressure on any building is calculated in accordance with the relevant British Standard (CP 3, Chapter V, Part II).

For this design example the basic pressure and local pressure intensities can be assumed to have been calculated as:

Dynamic wind pressure, q	$=0.65$ kN/m^2
C_{pe} on windward face	$=0.70$
C_{pe} on leeward face	$=0.60$
C_{pe} on roof	$=0.80$
C_{pi} on walls	$=$ either $+0.2$ or -0.3

Therefore, characteristic value wind loads are:

Pressure on windward wall	$=W_{k1}=(C_{pe}-C_{pi})q=(0.70+0.30)0.65$ $=0.65$ kN/m^2
Suction on leeward wall	$=W_{k2}=(C_{pe}-C_{pi})q=(0.60-0.20)0.65$ $=0.26$ kN/m^2
Gross wind uplift on roof	$=W_{k3}=(C_{pe}+C_{pi})q=(0.8+0.2)0.65$ $=0.65$ kN/m^2

(b) Dead and superimposed loads

Characteristic superimposed load $Q_k=0.75$ kN/m^2 (no general access to roof)

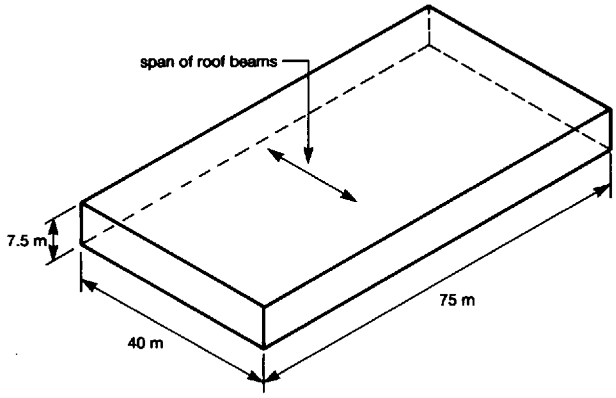

Figure 34: Overall dimensions for Worked Example 1

Characteristic dead load G_k assumed to be

metal decking and insulation	$=0.165$
felt and chippings	$=0.285$
self weight of roof beams	$=0.200$
Total G_k	$=0.65$ kN/m^2

(c) Design loading

The critical loading for tall single-storey walls is usually dead + wind although the loading condition of dead + imposed + wind should also be checked.

Design dead load $=0.9G_k$ or $1.4G_k$

Design wind load $=1.4W_k$ or $0.015G_k$

By inspection, the most critical loading combination will be given by:

Design dead load $=0.9 \times 0.65=0.585$ kN/m^2

Design wind loads:

pressure W_{k1}	$=1.4 \times 0.65=0.91$ kN/m^2
suction W_{k2}	$=1.4 \times 0.26=0.364$ kN/m^2
uplift W_{k3}	$=1.4 \times 0.65=0.91$ kN/m^2
Resultant load on roof	$=0.585-0.91=-0.325$ kN/m^2 (i.e. uplift)

Stage 2. Select trial section

For a wall height of 7.5 m and a characteristic wind load of 0.65 kN/m^2, trial section coefficients $K_2=0.85$ kN/m and $Z=27.5 \times 10^{-3}$ m^3 are read from Figures 28 and 29 respectively.

Although Section D from Table 1 meets the requirements for K_2 and Z, it is less economic than Section E. Therefore select wall section E and analyse this section fully. The wall section to be analysed is shown in Figure 31 and comprises 550 mm overall thickness with bonded cross-ribs spaced at 1180 mm centres.

As noted earlier, in practice the bonded wall may require rendering to provide adequate resistance to rain penetration.

The properties of this wall section have been calculated as:

I per metre	$=0.0106$ m^4
Z per metre	$=0.038$ m^3
A per metre	$=0.230$ m^2
K_2 (20)	$=1.080$ kN per m height
K_1	$=21.24$ m^{-2}

Stage 3. Capping beam

The design wind uplift in this example exceeds the design dead load of the roof.

The main roof beams are spaced at 4.8 m centres and the uplift reaction at each roof beam bearing $=(4.8 \times 0.325)40/2=31.20$ kN.

The roof beam will be anchored to a continuous reinforced concrete capping beam, as shown in Figure