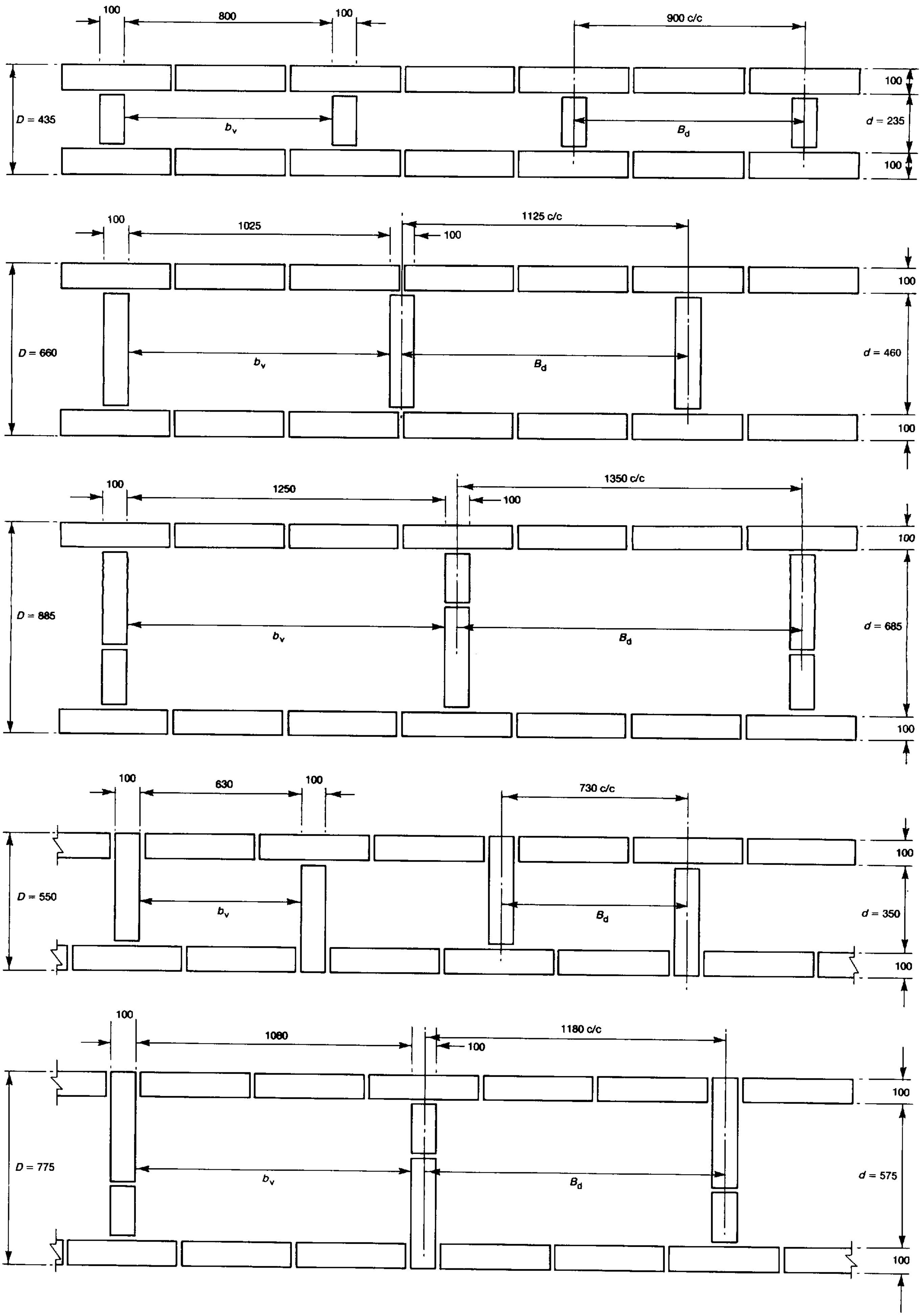

Figure 20: Wall depths and cross-rib spacings used for Table 1 data

2.3.2 *Concrete brickwork*

Figure 21 shows some typical bonded concrete brick diaphragm wall profiles which have been found useful in practice. They are based on the standard brick format. The calculations for the structural properties of these sections are made in the manner described in Section 2.3.1. Table 2 presents the properties of a number of such walls of varying depth and cross-rib spacing.

2.4 Assumed behaviour of a diaphragm wall

The critical loading condition for single-storey buildings is generally that created by lateral loading from wind forces. The vertical loading condition is rarely significant by itself and on the basis of engineering judgement may often be eliminated from the design.

The limiting stresses for lateral loading are usually on the tensile face of the wall. The geometric profile of the diaphragm wall is well suited to offering a high resistance to these limiting stresses.

For most buildings, and particularly tall single-storey buildings, the roof is designed and detailed to act as a prop or tie to the head of the external walls. This permits the wall design to be similar to that of a propped cantilever, which generally results in the maximum structural efficiency and therefore economy.

In a propped cantilever design approach there are two locations of critical bending moment to consider (see Figure 22):

level A – at the base of the wall (which is generally at dpc level) where it is recommended that the wall is designed as a cracked section.

level B – at a level approximately 3/8 h down from the top of the wall where the wall is designed as an uncracked section.

The resistance at these two levels of critical bending moment is provided by:

at level A – the stability moment of resistance (MR_s) of the cracked wall;

at level B – the flexural tensile resistance of the wall.

By treating level A as a 'cracked section' the design is unaffected by any lateral movement of the prop which may occur due to lateral deflection of the roof plate.

2.4.1 *Stability moment of resistance* (MR_s)

Single-storey buildings tend to have a lightweight roof construction and low superimposed roof loading. Hence, the forces and moments due to lateral wind pressure have greater effect on the stresses in the supporting masonry than they do in multi-storey buildings. Since there is little precompression, the wall's stability relies more on its own gravitational mass (including any net roof loads) and the resulting resistance moment. Under lateral wind pressure loading, the wall will tend to rotate at dpc level on its leeward face and 'crack' at the same level on the windward face as indicated in Figure 23. The resistance moment is generated by the self weight of the wall (together with any net roof loads) acting over a lever arm of approximately half the thickness of the wall from the centre of rotation.

In limit state design the centre of rotation is taken as the centroid of a rectangular stress block and not as a knife edge, i.e. the extreme edge of the wall. Thus the lever arm is calculated as the distance between this centroid and the resultant of the downward vertical loads.

The width of the rectangular stress block is calculated as the width of masonry as shown in Figure 24, stressed to ultimate. This produces the *maximum* lever arm to generate the *maximum* stability moment of resistance MR_s.

It is convenient to approximate this lever arm value for the purpose of obtaining an initial trial section and, for this purpose only, a lever arm value of 0.475D has been found from experience to be a reasonable approximation. This approximate lever arm is shown in Figure 25 and its application is discussed in Section 2.4.5.

Table 2. Section properties – concrete bricks of work size 215 x 102.5 x 65 mm.

	Dimensions (m)				Section properties per diaphragm			Section properties per metre			Shear stress coefficient	Stability moment K_2 (kN/m)	
Section	D	d	B_d	b_v	$I \times 10^{-3}$ (m^4)	$Z \times 10^{-3}$ (m^3)	A (m^2)	$I \times 10^{-3}$ (m^4)	$Z \times 10^{-3}$ (m^3)	A (m^2)	K_1 (m^{-2})	K_2 (15)	K_2 (18)
1	0.44	0.235	1.4625	1.36	8.91	40.49	0.324	6.09	27.69	0.222	27.74	0.835	0.752
2	0.44	0.235	1.2375	1.135	7.55	34.32	0.278	6.10	27.73	0.225	27.66	0.846	0.762
3	0.44	0.235	1.0125	0.91	6.21	28.83	0.232	6.13	27.88	0.229	27.51	0.862	0.776
4	0.5575	0.352	1.4625	1.36	16.18	58.04	0.377	11.06	39.69	0.230	20.52	1.097	0.987
5	0.5575	0.352	1.2375	1.135	13.74	49.29	0.290	11.10	39.83	0.234	20.44	1.116	1.004
6	0.5575	0.352	1.0125	0.91	11.31	40.57	0.244	11.17	40.07	0.241	20.34	1.149	1.034
7	0.665	0.46	1.4625	1.36	24.81	74.62	0.347	16.96	51.02	0.237	16.56	1.348	1.212
8	0.665	0.46	1.2375	1.135	21.12	63.52	0.301	17.07	51.33	0.243	16.46	1.382	1.243
9	0.665	0.46	1.0125	0.91	17.43	52.43	0.254	17.21	51.77	0.251	16.37	1.427	1.284
10	0.7825	0.5775	1.4625	1.36	36.56	93.45	0.359	24.99	63.90	0.245	13.60	1.639	1.478
11	0.7825	0.5775	1.2375	1.135	31.18	79.69	0.313	25.19	64.40	0.253	13.49	1.693	1.523
12	0.7825	0.5775	1.0125	0.91	25.82	66.01	0.267	25.50	65.20	0.264	13.33	1.766	1.590
13	0.89	0.685	1.4625	1.36	49.46	111.14	0.37	33.82	76.00	0.253	11.64	1.925	1.733
14	0.89	0.685	1.2375	1.135	42.4	95.3	0.324	34.26	77.01	0.262	11.49	1.994	1.794
15	0.89	0.685	1.0125	0.91	34.86	78.34	0.278	34.43	77.37	0.274	11.44	2.085	1.877

Note: For Sections 1, 4, 7, 10, 13 the flange length slightly exceeds the limitations given in Clause 36.4.3(b) BS 5628. These sections have been included since they are the closest brick sizes to the flanges recommended in the code. If the designer is concerned at the marginal variation he may calculate the section properties on the basis of an effective flange width of 1.33 m.