(3) The length of any intersecting wall, which may be considered to act as a flange (see figure 5.6), is the thickness of the shear wall plus, on each side of it - where appropriate - the least of:

— $h_{tot}/5$, where h_{tot} is the overall height of the shear wall;

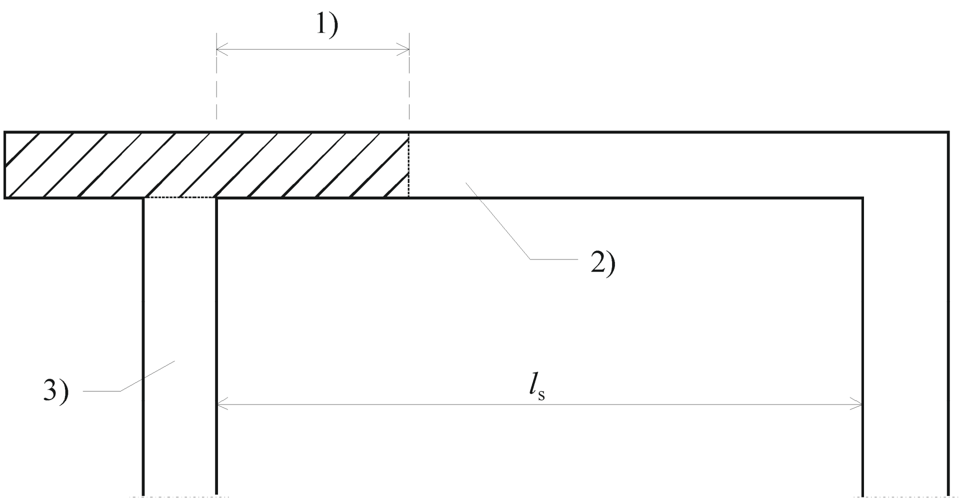
— half the distance between shear walls (l_s), when connected by the intersecting wall;

— the distance to the end of the wall;

— half the clear height (h);

— six times the thickness of the intersecting wall, t.

(4) In intersecting walls, openings with dimensions smaller than h/4 or l/4 may be disregarded. Openings with dimensions greater than h/4 or l/4 should be regarded as marking the end of the wall.



Key

1) the lesser of $\begin{cases} h_{tot}/5 \\ l_s/2 \\ h/2 \\ 6\,t \end{cases}$

2) intersecting wall

3) shear wall

Figure 5.6 — Flange widths that can be assumed for shear walls

(5) If the floors can be idealised as rigid diaphragms, the horizontal forces may be distributed to the shear walls in proportion to their stiffness.

(6)P Where the plan arrangement of the shear walls is asymmetric, or for any other reason the horizontal force is eccentric to the overall stiffness centre of the structure, account shall be taken of the effect of the consequent rotation on the individual walls (torsional effects).

(7) If the floors are not sufficiently rigid when considered as horizontal diaphragms (for example, precast concrete units which are not inter-connected) horizontal forces to be resisted by the shear walls should be taken to be the forces from the floors to which they are directly connected, unless a semi rigid analysis is carried out.

(8) The maximum horizontal load on a shear wall may be reduced by up to 15 % provided that the load on the parallel shear walls is correspondingly increased.

(9) When deriving the relevant design load that assists shear resistance, the vertical load applied to slabs spanning in two directions may be distributed equally onto the supporting walls; in the case of floor or roof slabs spanning one way, a 45° spread of the load may be considered in deriving the axial load, at the lower storeys, on the walls not directly loaded.

(10) The distribution of shear stress along the compressed part of a wall may be assumed to be constant.

5.5.4 Reinforced masonry members subjected to shear loading

(1) In calculating the design shear load in reinforced masonry members with uniformly distributed loading, it may be assumed that the maximum shear load occurs at a distance $d/2$ from the face of a support, where d is the effective depth of the member.

(2) When taking the maximum shear load at $d/2$ from the face of a support, the following conditions should be satisfied:

— the loading and support reactions are such that they cause diagonal compression in the member (direct support);

— at an end support, the tension reinforcement required at a distance 2,5 d from the face of the support is anchored into the support;

— at an intermediate support, the tension reinforcement required at the face of the support extends for a distance at least 2,5 d, plus the anchorage length, into the span.

5.5.5 Masonry walls subjected to lateral loading

(1) When analysing masonry walls subjected to lateral loading, allowance should be made in the design for the following:

— the effect of damp proof courses;

— support conditions and continuity over supports.

(2) A faced wall should be analysed as a single-leaf wall constructed entirely of the units giving the lower flexural strength.

(3) A movement joint in a wall should be treated as an edge across which moment and shear may not be transmitted.

NOTE Some specialised anchors are designed to transmit moment and/or shear across a movement joint; their use is not covered in this standard.

(4) The reaction along an edge of a wall due to the load may be assumed to be uniformly distributed when designing the means of support. Restraint at a support may be provided by ties, by bonded masonry returns or by floors or roofs.

(5) Where laterally loaded walls are bonded (see 8.1.4) to vertically loaded walls, or where reinforced concrete floors bear onto them, the support may be considered as being continuous. A damp-proof course should be considered as providing simple support. Where walls are connected to a vertically load bearing wall or other suitable structure by ties at the vertical edges, partial moment continuity at the vertical sides of the wall may be assumed, if the strength of the ties is verified to be sufficient.

(6) In the case of cavity walls, full continuity may be assumed even if only one leaf is continuously bonded across a support, provided that the cavity wall has ties in accordance with 6.3.3. The load to be transmitted from a wall to its support may be taken by ties to one leaf only, provided that there is adequate connection between the two leaves (see 6.3.3) particularly at the vertical edges of the walls. In all other cases, partial continuity may be assumed.

(7) When the wall is supported along 3 or 4 edges, the calculation of the applied moment, M_{Edi}, may be taken as:

— when the plane of failure is parallel to the bed joints, i. e. in the f_{xk1} direction:

$M_{\mathrm{Ed1}} = \alpha_1 W_{\mathrm{Ed}} l^2$ per unit length of the wall (5.17)

or,

— when the plane of failure is perpendicular to the bed joints, i. e. in the f_{xk2} direction:

$M_{\mathrm{Ed2}} = \alpha_2 W_{\mathrm{Ed}} l^2$ per unit height of the wall (5.18)

where

α_1, α_2 are bending moment coefficients taking account of the degree of fixity at the edges of the walls, the height to length ratio of the walls; they can be obtained from a suitable theory;

l is the length of the wall;

W_{Ed} is the design lateral load per unit area.

NOTE Values of the bending coefficient α_1 and α_2 may be obtained from Annex E for single leaf walls with a thickness less than or equal to 250 mm, where $\alpha_1 = \mu\, \alpha_2$
where:
μ is the orthogonal ratio of the design flexural strengths of the masonry,
$f_{\mathrm{xd1}}/f_{\mathrm{xd2}}$, see 3.6.4 or $f_{\mathrm{xd1,app}}/f_{\mathrm{xd2}}$, see AC) 6.3.1(4) (AC or $f_{\mathrm{xd1}}/f_{\mathrm{xd2,app}}$, see AC) 6.5.2(9) (AC;

(8) The bending moment coefficient at a damp proof course may be taken as for an edge over which full continuity exists when the design vertical stress on the damp proof course equals or exceeds the design tensile stress caused by the moment arising due to the action.

(9) When the wall is supported only along its bottom and top edges, the applied moment may be calculated from normal engineering principles, taking into account any continuity.

(10) AC) In a laterally loaded panel or free standing wall built of masonry set in mortar designations M2 to M20, and designed in accordance with 6.3, the dimensions should be limited to avoid undue movements resulting from deflections, creep, shrinkage, temperature effects and cracking.

NOTE The limiting values may be obtained from Annex F. (AC

(11) When irregular shapes of walls, or those with substantial openings, are to be designed, an analysis, using a recognized method of obtaining bending moments in flat plates, for example, finite element method or yield line analogy may be used, taking into account the anisotropy of masonry when appropriate.

Section 6 Ultimate Limit State

6.1 Unreinforced masonry walls subjected to mainly vertical loading

6.1.1 General

(1)P The resistance of masonry walls to vertical loading shall be based on the geometry of the wall, the effect of the applied eccentricities and the material properties of the masonry.

(2) In calculating the vertical resistance of masonry walls, it may be assumed that:

— plane sections remain plane;

— the tensile strength of masonry perpendicular to bed joints is zero.

6.1.2 Verification of unreinforced masonry walls subjected to mainly vertical loading

6.1.2.1 General

(1)P At the ultimate limit state, the design value of the vertical load applied to a masonry wall, N_{Ed}, shall be less than or equal to the design value of the vertical resistance of the wall, N_{Rd}, such that:

$$N_{Ed} \leq N_{Rd} \tag{6.1}$$

(2) The design value of the vertical resistance of a single leaf wall per unit length, N_{Rd}, is given by:

$$N_{Rd} = \Phi \, t \, f_d \tag{6.2}$$

where

Φ is the capacity reduction factor, Φ_i, at the top or bottom of the wall, or Φ_m, in the middle of the wall, as appropriate, allowing for the effects of slenderness and eccentricity of loading, obtained from 6.1.2.2;

t is the thickness of the wall;

f_d is the design compressive strength of the masonry, obtained from 2.4.1 and 3.6.1.

(3) Where the cross-sectional area of a wall is less than 0,1 m^2, the design compressive strength of the masonry, f_d, should be multiplied by the factor:

$$(0,7 + 3\,A) \tag{6.3}$$

where

A is the loaded horizontal gross cross-sectional area of the wall, expressed in square metres.

(4) For cavity walls, each leaf should be verified separately, using the plan area of the loaded leaf and the slenderness ratio based upon the effective thickness of the cavity wall, calculated according to equation (5.11).

(5) A faced wall, should be designed in the same manner as a single-leaf wall constructed entirely of the weaker units, using the value of K, from table 3.3, appropriate to a wall with a longitudinal mortar joint.

(6) A double-leaf wall, tied together according to clause 6.5 may be designed as a single-leaf wall, if both leaves have a load of similar magnitude, or, alternatively, as a cavity wall.

(7) When chases or recesses are outside the limits given in clause 8.6, the effect on loadbearing capacity should be taken into account as follows:

— vertical chases or recesses should be treated either as a wall end or, alternatively, the residual thickness of the wall should be used in the calculations of the design vertical load resistance;

— horizontal or inclined chases should be treated by verifying the strength of the wall at the chase position, taking account of the load eccentricity.

NOTE As a general guide the reduction in vertical loadbearing capacity may be taken to be proportional to the reduction in cross-sectional area due to any vertical chase or recess, provided that the reduction in area does not exceed 25 %.

6.1.2.2 Reduction factor for slenderness and eccentricity

(1) The value of the reduction factor for slenderness and eccentricity, Φ, may be based on a rectangular stress block as follows:

(i) At the top or bottom of the wall (Φ_i)

$$\Phi_i = 1 - 2\frac{e_i}{t} \qquad (6.4)$$

where

e_i is the eccentricity at the top or the bottom of the wall, as appropriate, calculated using the equation (6.5):

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0{,}05\, t \qquad (6.5)$$

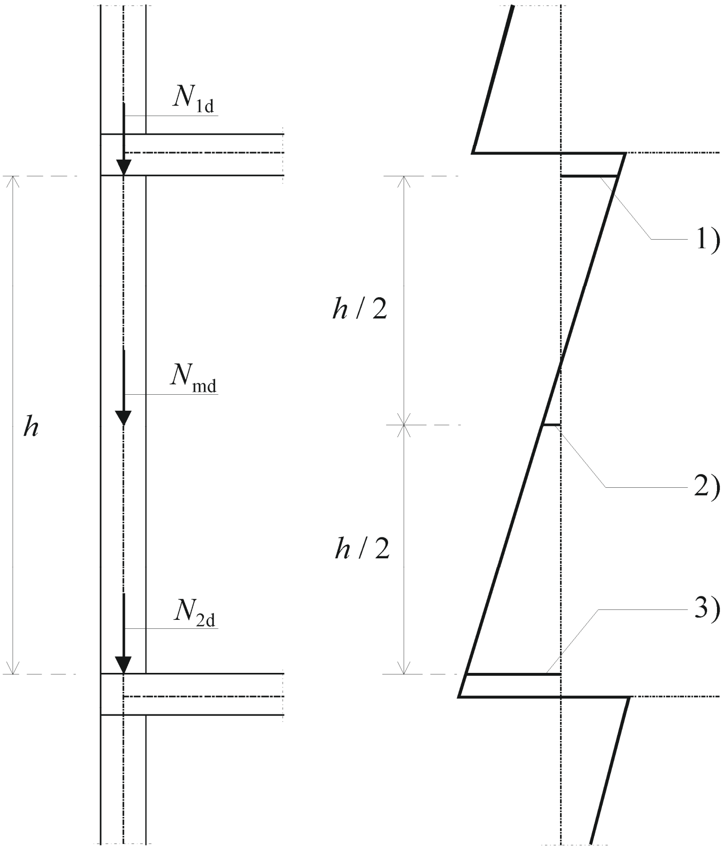
M_{id} is the design value of the bending moment at the top or the bottom of the wall resulting from the eccentricity of the floor load at the support, analysed according to 5.5.1 (see figure 6.1);

N_{id} is the design value of the vertical load at the top or bottom of the wall;

e_{he} is the eccentricity at the top or bottom of the wall, if any, resulting from horizontal loads (for example, wind);

e_{init} AC) is the initial eccentricity with a sign that increases the absolute value of e_i (see 5.5.1.1) (AC;

t is the thickness of the wall.



Key

1) M_{1d} (at underside of floor)
2) M_{md} (at mid height of wall)
3) M_{2d} (at top of floor)

Figure 6.1 — Moments from calculation of eccentricities

(ii) In the middle of the wall height (Φ_m)

By using a simplification of the general principles given in 6.1.1, the reduction factor within the middle height of the wall Φ_m, may be determined [AC) *deleted text* (AC] using e_{mk}, where:

e_{mk} is the eccentricity at the middle height of the wall, calculated using equations (6.6) and (6.7):

$$e_{mk} = e_m + e_k \geq 0{,}05\, t \qquad (6.6)$$

[AC) $$e_m = \frac{M_{md}}{N_{md}} + e_{hm} + e_{init} \qquad (6.7)$$ (AC]

e_m is the eccentricity due to loads;

M_{md} is the design value of the greatest moment at the middle of the height of the wall resulting from the moments at the top and bottom of the wall (see figure 6.1), including any load applied eccentrically to the face of the wall (e. g. brackets);

N_{md} is the design value of the vertical load at the middle height of the wall, including any load applied eccentrically to the face of the wall (e. g. brackets);

e_{hm} is the eccentricity at mid-height resulting from horizontal loads (for example, wind);

NOTE The inclusion of e_{hm} depends on the load combination being used for the verification; its sign relative to that of M_{md}/N_{md} should be taken into account.

e_{init} AC⟩ is the initial eccentricity a sign that increases the absolute value of e_m (see 5.5.1.1) ⟨AC;

h_{ef} is the effective height, obtained from 5.5.1.2 or the appropriate restraint or stiffening condition;

t_{ef} is the effective thickness of the wall, obtained from 5.5.1.3;

e_k is the eccentricity due to creep, calculated from the equation (6.8):

$$e_k = 0{,}002\, \varphi_\infty \frac{h_{ef}}{t_{ef}} \sqrt{t\, e_m} \tag{6.8}$$

ϕ_∞ is the final creep coefficient (see note under 3.7.4(2))

AC⟩ NOTE Φ_m, may be determined from Annex G, using e_{mk} as expressed above. ⟨AC

(2) For walls having a slenderness ratio of λ_c or less, the creep eccentricity, e_k may be taken as zero.

NOTE The value of λ_c to be used in a country may be found in its National Annex, the recommended value of λ_c is 15. The country can make a distinction for different types of masonry related to the national choices made on the final creep coefficient.

6.1.3 Walls subjected to concentrated loads

(1)P The design value of a concentrated vertical load, N_{Edc}, applied to a masonry wall, shall be less than or equal to the design value of the vertical concentrated load resistance of the wall, N_{Rdc}, such that

$$N_{Edc} \leq N_{Rdc} \tag{6.9}$$

(2) When a wall, built with Group 1 masonry units and detailed in accordance with section 8, other than a shell bedded wall, is subjected to a concentrated load, the design value of the vertical load resistance of the wall is given by:

$$N_{Rdc} = \beta\, A_b\, f_d \tag{6.10}$$

where

$$\beta = \left(1 + 0{,}3\frac{a_1}{h_c}\right)\left(1{,}5 - 1{,}1\frac{A_b}{A_{ef}}\right) \tag{6.11}$$

which should not be less than 1,0 nor taken to be greater than:

$1{,}25 + \dfrac{a_1}{2\, h_c}$ or 1,5 whichever is the lesser

where

β is an enhancement factor for concentrated loads;

a_1 is the distance from the end of the wall to the nearer edge of the loaded area (see figure 6.2);

h_c is the height of the wall to the level of the load;

A_b is the loaded area;

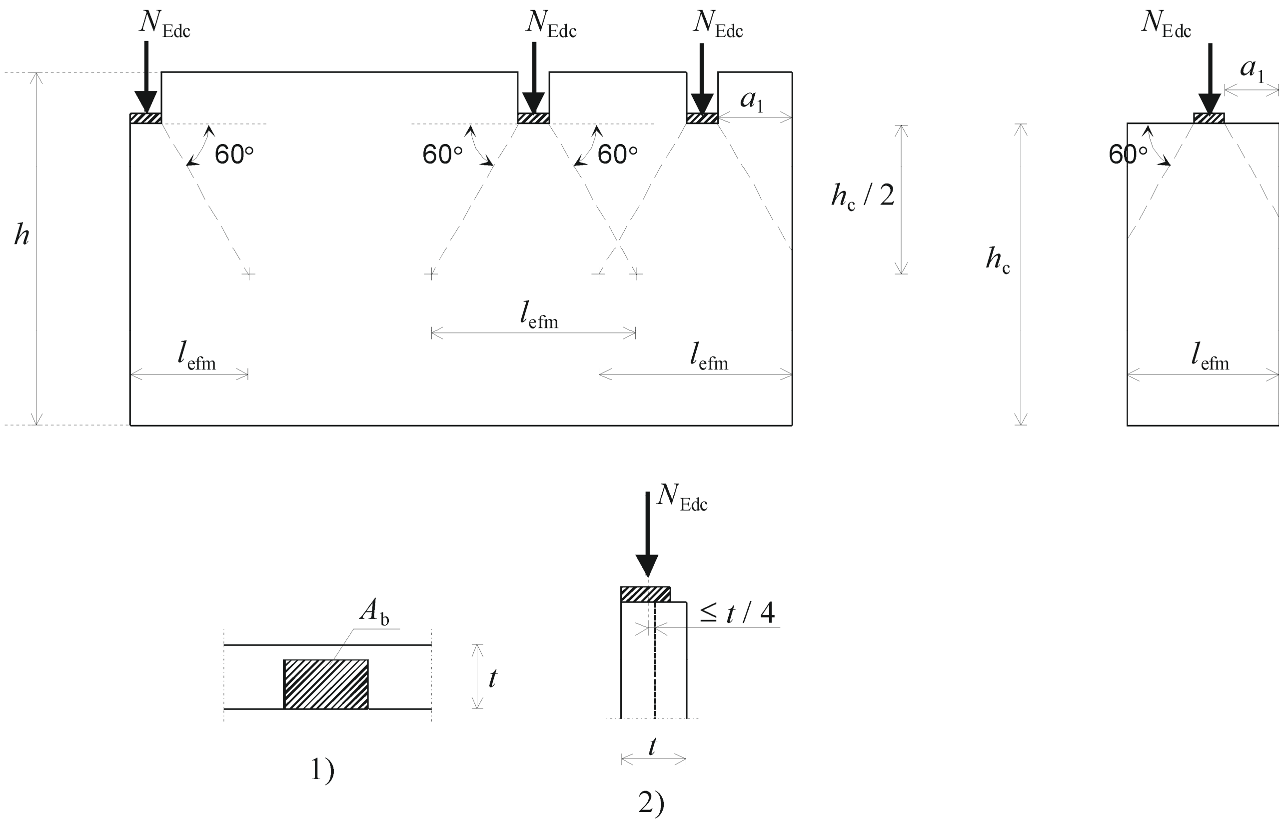
A_{ef} is the effective area of bearing, i. e. $l_{efm} \cdot t$;

l_{efm} is the effective length of the bearing as determined at the mid height of the wall or pier (see figure 6.2)~~;~~

t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm;

$\frac{A_b}{A_{ef}}$ is not to be taken greater as 0,45.

NOTE Values for the enhancement factor for β are shown in graphical form in Annex H.



Key

1) plan

2) section

Figure 6.2 — Walls subjected to concentrated load

(3) For walls built with Groups 2, 3 and Group 4 masonry units and when shell bedding is used, it should be verified that, locally under the bearing of a concentrated load, the design compressive stress does not exceed the design compressive strength of masonry, f_d (i.e. β is taken to be AC) 1,0 (AC).

(4) The eccentricity of the load from the centre line of the wall should not be greater than $t/4$ (see figure 6.2).

(5) In all cases, the requirements of 6.1.2.1 should be met at the middle height of the wall below the bearings, including the effects of any other superimposed vertical loading, particularly for the case where concentrated loads are sufficiently close together for their effective lengths to overlap.

(6) The concentrated load should bear on a Group 1 unit or other solid material of length equal to the required bearing length plus a length on each side of the bearing based on a 60° spread of load to the base of the solid material; for an end bearing the additional length is required on one side only.

(7) Where the concentrated load is applied through a spreader beam of adequate stiffness and of width equal the thickness of the wall, height greater than 200 mm and length greater than three times the bearing length of the load, the design value of the compressive stress beneath the concentrated load should not exceed 1,5 f_d.

6.2 Unreinforced masonry walls subjected to shear loading

(1)P At the ultimate limit state the design value of the shear load applied to the masonry wall, V_{Ed}, shall be less than or equal to the design value of the shear resistance of the wall, V_{Rd}, such that :

$$V_{Ed} \leq V_{Rd} \tag{6.12}$$

(2) The design value of the shear resistance is given by:

$$V_{Rd} = f_{vd}\, t\, l_c \tag{6.13}$$

[A1) or as an alternative by:

$$V_{Rd} = V_{Rdlt} \tag{6.14}$$ (A1]

where

f_{vd}	is the design value of the shear strength of masonry, obtained from 2.4.1 and 3.6.2, based on the average of the vertical stresses over the compressed part of the wall that is providing the shear resistance;
t	is the thickness of the wall resisting the shear;
l_c	is the length of the compressed part of the wall, ignoring any part of the wall that is in tension;
[A1) V_{Rdlt}	is the design value of the limiting shear resistance

NOTE The decision to use equation (6.13) or (6.14) in a country, and the values or derivation of V_{Rdlt} related to e.g. the tensile strength of the units and/or overlap in the masonry, if that option is chosen, may be found in its National Annex. If no choice is given, equation (6.13) should be used. (A1]

(3) The length of the compressed part of the wall, l_c, should be calculated assuming a linear stress distribution of the compressive stresses, and taking into account any openings, chases or recesses; any portion of the wall subjected to vertical tensile stresses should not be used in calculating the area of the wall to resist shear.

(4)P The connections between shear walls and flanges of intersecting walls shall be verified for vertical shear.

(5) The length of the compressed part of the wall should be verified for the vertical loading applied to it and the vertical load effect of the shear loads.

6.3 Unreinforced masonry walls subjected to lateral loading

6.3.1 General

(1)P At the ultimate limit state, the design value of the moment applied to the masonry wall, M_{Ed} (see 5.5.5), shall be less than or equal to the design value of the moment of resistance of the wall, M_{Rd}, such that:

$$M_{Ed} \leq M_{Rd} \tag{6.15}$$

(2) The orthogonal strength ratio, μ, of the masonry should be taken into account in the design.

(3) The design value of the lateral moment of resistance of a masonry wall, M_{Rd}, per unit height or length, is given by:

$$M_{Rd} = f_{xd} Z \tag{6.16}$$

where

f_{xd} is the design flexural strength appropriate to the plane of bending, obtained from 3.6.4, 6.3.1(4) or 6.6.2 (9);

Z is the elastic section modulus of unit height or length of the wall.

(4) When a vertical load is present, the favourable effect of the vertical stress may be taken into account either by:

(i) using the apparent flexural strength, $f_{xd1,app}$, given by equation (6.17), the orthogonal ratio used in (2) above being modified accordingly.

$$f_{xd1,app} = f_{xd1} + \sigma_d \tag{6.17}$$

where

f_{xd1} is the design flexural strength of masonry with the plane of failure parallel to the bed joints, see 3.6.4;

σ_d is the design compressive stress on the wall, not taken [A1> to be greater than 0,15 N_{Rd} in the middle of the wall according to 6.1.2.1(2) <A1]

or

(ii) by calculating the resistance of the wall using formula (6.2) in which Φ is replaced by Φ_{fl}, taking into account the flexural strength, f_{xd1}.

NOTE This Part does not include a method of calculating ϕ_{fl} including flexural strength.

(5) In assessing the section modulus of a pier in a wall, the outstanding length of flange from the face of the pier should be taken as the lesser of:

— h/10 for walls spanning vertically between restraints;

— h/5 for cantilever walls;

— half the clear distance between piers;