

BUILDINGS IN FIRED-BRICK AND OTHER MASONRY UNITS

4.1 INTRODUCTION

The buildings in fired bricks, solid concrete blocks and hollow concrete or mortar blocks are dealt with in this chapter. The general principles and most details of earthquake resistant design and construction of brick-buildings are applicable to those using other rectangular masonry units such as solid blocks of mortar, concrete, or stabilized soil, or hollow blocks of mortar, or concrete having adequate compressive strength. Some construction details only differ for hollow blocks, which are also indicated as necessary.

4.2 TYPICAL DAMAGE AND FAILURE OF MASONRY BUILDINGS

The creation of tensile and shearing stresses in walls of masonry buildings is the primary cause of different types of damage suffered by such buildings. The typical damages and modes of failure are briefly described below:

4.2.1 Non-structural damage

The non-structural damage is that due to which the strength and stability of the building is not affected. Such damage occurs very frequently even under moderate intensities of earthquakes:

- Cracking and overturning of masonry parapets, roof chimney, large cantilever cornices and balconies.
- Falling of plaster from walls and ceiling particularly where it was loose.
- Cracking and overturning of partition walls, filler walls and cladding walls from inside of frames. (Though not usually accounted for in calculations, this type of damage reduced the lateral strength of the building).
- Cracking and failing of ceilings.
- Cracking of glass panes.
- Failing of loosely placed objects, overturning of cupboards, etc.

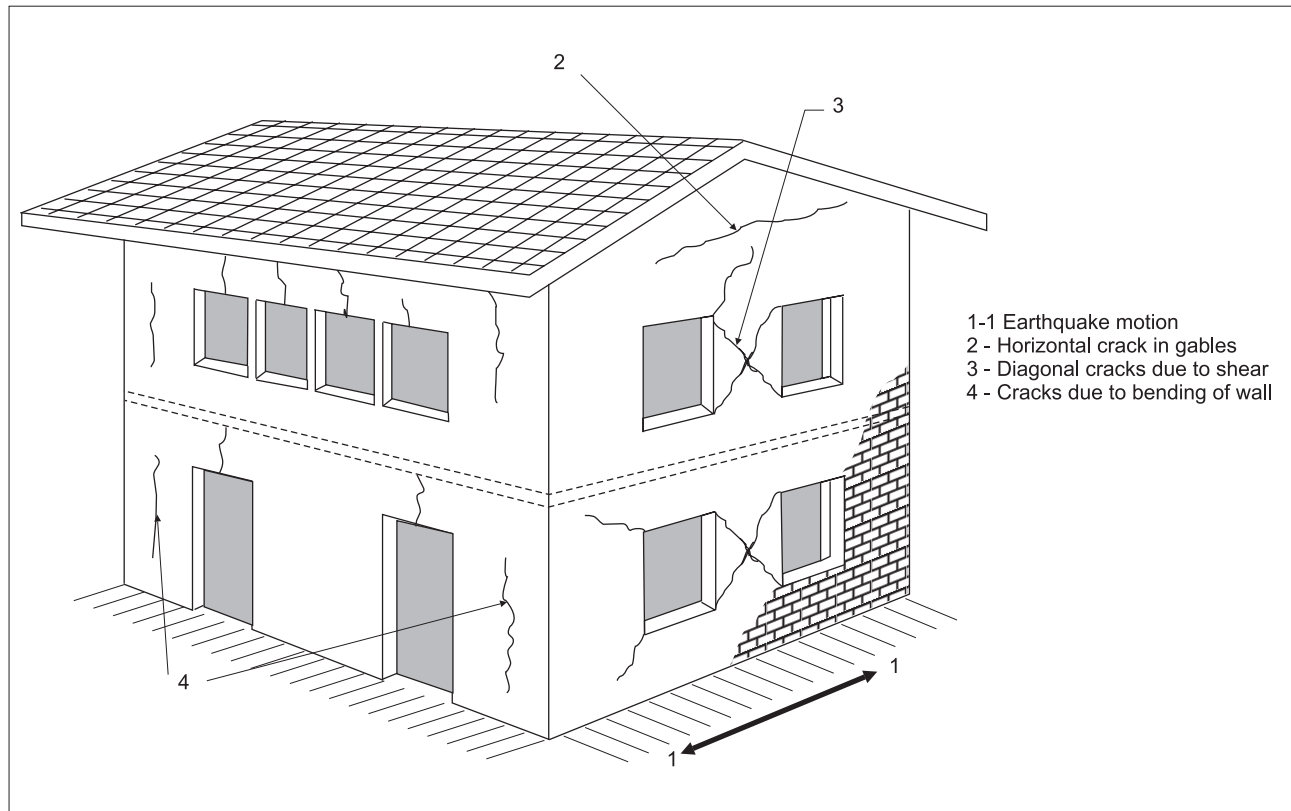


Fig 4.1 Cracking in bearing wall building due to bending and shear

4.2.2 Damage and failure of bearing walls

- (i) Failure due to racking shear is characterized by diagonal cracks which could be due to diagonal compression or diagonal tension. Such failure may be either through the pattern of joints or diagonally through masonry units. These cracks usually initiate at the corner of openings and sometimes at centre of wall segment. This kind of failure can cause partial or complete collapse of the structure, Fig 4.1.
- (ii) A wall can fail as a bending member loaded by seismic inertia forces on the mass of the wall itself in a direction, transverse to the plane of the

wall. Tension cracks occur vertically at the centre, ends or corners of the walls. Longer the wall and longer the openings, more prominent is the damage, Fig 4.1. Since earthquake effects occur along both axes of a building simultaneously, bending and shearing effects occur often together and the two modes of failures are often combined. Failure in the piers occur due to combined action of flexure and shear.

- (iii) Unreinforced gable end masonry walls are very unstable and the strutting action of purlins imposes additional force to cause their failure. Horizontal bending tension cracks are caused in the gables.

(iv) The deep beam between two openings one above the other is a weak point of the wall under lateral inplane forces. Cracking in this zone occurs before diagonal cracking of piers, Fig 4.2. In order to prevent it and to enable the full distribution of shear among all piers, either a rigid slab or RC band must exist between them.

(v) Walls can be damaged due to the seismic force of the roof, which can cause the formation of tension cracks and separation of supporting walls, Fig 4.3. This mode of failure is the characteristic of massive flat roofs (or floors) supported by joists, which in turn are supported by bearing walls, but without proper connection with them. Also if the connection with foundation is not adequate, walls crack there and slide. This may cause failure of plumbing pipes too.

(vi) Failure due to torsion and warping: The damage in unsymmetrical building occurs due to torsion and warping in an earthquake, Fig 3.1. This mode of failure causes excessive cracking due to shear in all walls. Larger damage occurs near the corner of the building.

(vii) Arches across openings in walls are often badly cracked since the arches tend to lose their end thrust under in-plane shaking of walls.

(viii) Under severe prolonged intense ground motions, the following happens:

- the cracks become wider and the masonry units become loose

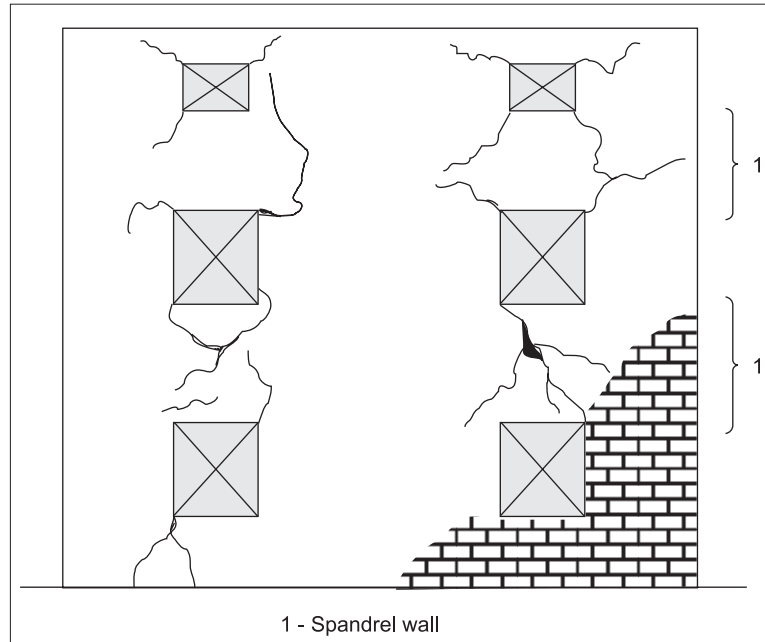


Fig 4.2 Cracking of spandrel wall between opening

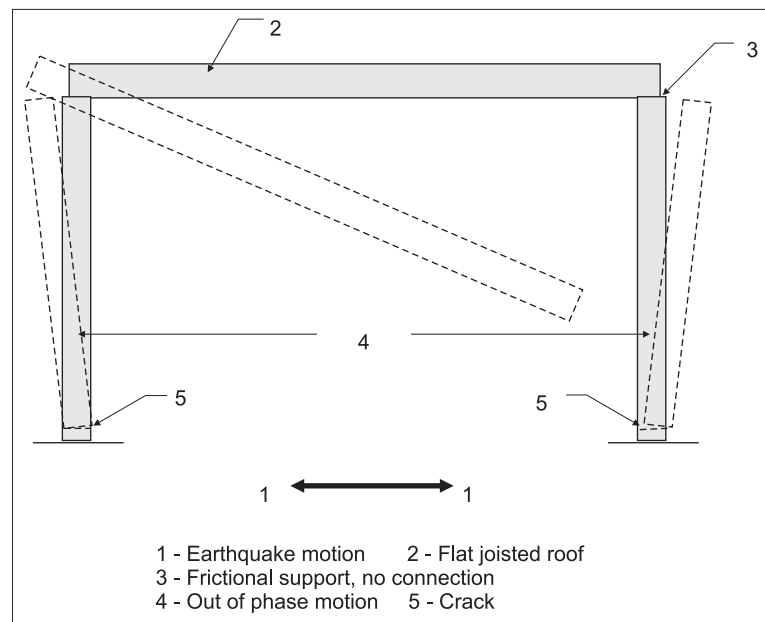


Fig 4.3 Fall of roof because of inadequate connection between roof and wall

- partial collapse and gaps in walls occur due to falling of loose masonry units, particularly at location of piers.
- falling of spandrel masonry due to collapse of piers

- falling of gable masonry due to out of plane cantilever action
- walls get separated at corners and intermediate T-junctions and fall outwards.
- roof collapse, either partial or full
- certain types of roofs may slide off the top of walls and the roof beams fall down
- masonry arches across wall openings as well as those used for roof collapse completely.

4.2.3 Failure of ground

- (i) Inadequate depth of foundation: Shallow foundations deteriorate as a result of weathering and consequently become weak for earthquake resistance.
- (ii) Differential settlement of foundation: During severe ground shaking, liquefaction of loose water-saturated sands and differential compaction of weak loose soils occur which lead to excessive cracking and tilting of buildings which may even collapse completely.
- (iii) Sliding of slopes: Earthquakes cause sliding failures in man-made as well as natural hill slopes and any building resting on such a slope have a danger of complete disastrous disintegration.

4.2.4 Failure of roofs and floors

- (i) Dislodging of roofing material: Improperly tied roofing material is dislodged due to inertia forces acting on the roof. This mode of failure is

typical of sloping roofs, particularly when slates, clay, tiles etc. are used as roofing material.

Brittle material like asbestos cement may be broken if the trusses and sheeting purlins are not properly braced together.

- (ii) Weak roof to support connection is the cause of separation of roof truss from supports, although complete roof collapse mostly occurs due to collapse of supporting structure. The rupture of bottom chord of roof truss may cause a complete collapse of truss as well as that of walls, Fig 4.4.

- (iii) Heavy roofs as used in rural areas with large thickness of earth over round timbers cause large inertia forces on top of walls and may lead to complete collapse in severe earthquake shocks.

- (iv) Lean-to roofs easily cause instability in the lower supporting walls or piers and collapse easily due to lack of ties.

4.2.5 Causes of damage in masonry buildings

The following are the main weaknesses in the materials and unreinforced masonry constructions and other reasons for the extensive damage of such buildings:

- Heavy weight and very stiff buildings, attracting large seismic inertia forces.
- Very low tensile strength, particularly with poor mortars.
- Low shear strength, particularly with poor mortars.

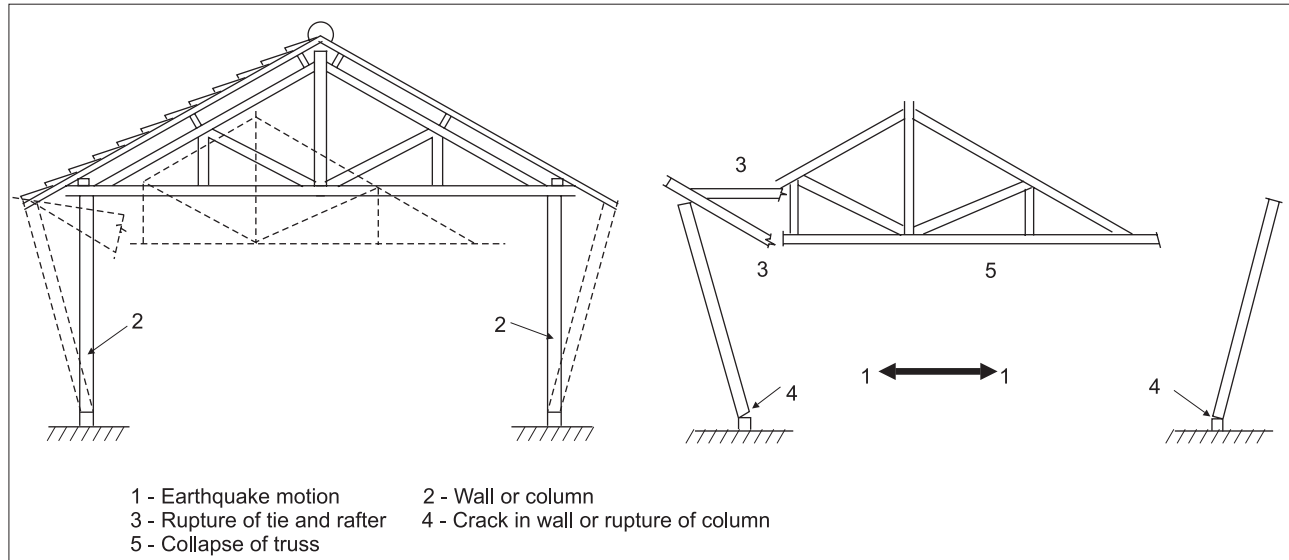


Fig 4.4 Failure due to rupture of bottom chord of roof truss

- Brittle behaviour in tension as well as compression.
 - Weak connection between wall and wall.
 - Stress concentration at corners of windows and doors.
 - Overall unsymmetry in plan and elevation of building.
 - Unsymmetry due to imbalance in the sizes and positions of openings in the walls.
 - Defects in construction such as use of substandard materials, unfilled joints between bricks, not-plumb walls, improper bonding between walls at right angles, etc.
- (ii) Mix of the mortar used and age at which tested. The mortar used for different wall constructions varies in quality as well as strength. It is generally described on the basis of the main binding material such as cement or lime mortar, cement lime composite mortar, lime-pozzolana or hydraulic lime mortar. Clay mud mortar is also used in many countries particular in rural areas.
- (iii) Slenderness ratio of the wall, that is, smaller of the ratio of effective height and effective length of the wall to its thickness. Larger is the slenderness ratio, smaller the strength.
- (iv) Eccentricity of the vertical toad on the wall- Larger the eccentricity, smaller the strength.

4.2 TYPICAL STRENGTHS OF MASONRY

The crushing strength of masonry used in the position of walls depends on many factors such as the following:

- (i) Crushing strength of the masonry unit.

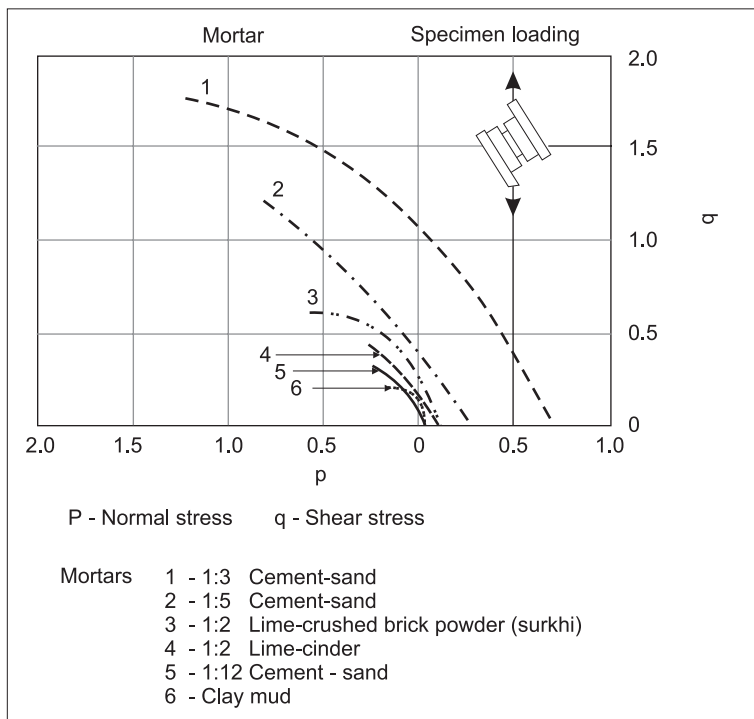
- (v) Percentage of openings in the wall — larger the openings, smaller the strength. The tensile and shearing strengths of masonry mainly depend upon the bond or adhesion at the contact surface between the masonry

Table 4.1 Typical strengths of masonry

Mortar mix		Tensile strength, MPa	Shearing strength, MPa	Compressive strength in MPa corresponding to crushing strength of masonry unit			
cement	sand			3.5	7.0	10.5	14.0
1	12	0.04	0.22	1.5	2.4	3.3	3.9
1	6	0.25	0.39	2.1	3.3	5.1	6.0
1	3	0.71	1.04	2.4	4.2	6.3	7.5

unit and the mortar and, in general, their values are only a small percentage of the crushing strength. Richer is a mortar in cement or lime content, higher is the percentage of tensile and shearing strength in relation to the crushing strength. Test carried out on brick-couplets using hand made bricks in cement mortar give the typical values as shown in Table 4.1.

Brick couplet tests under combined tension-shear and compression-shear stresses

**Fig 4.5 Combined stress couplet test results**

show that the shearing strength decreases when acting with tension and increases when acting with compression. Fig 4.5 shows the combined strengths.

The tensile strength of masonry is not generally relied upon for design purposes under normal loads and the area subjected to tension is assumed cracked. Under seismic conditions, it is recommended that the permissible tensile and shear stresses on the area of horizontal mortar bed joint in masonry may be adopted as given in Table 4.2.

The modulus of elasticity of masonry very much depends upon the density and stiffness of masonry unit, besides the mortar mix. For brickwork the values are of the order 2000 MPa for cement-sand mortar in 1:6 proportion. The mass density of masonry mainly depends on the type of masonry unit. For example brickwork will have a mass density of about 1900 kg/m³ and dressed stone masonry 2400 kg/m³.

The slenderness ratio of the wall is taken as the lesser of h/t and l/t where h = effective height of the wall and L = its effective length. The allowable stresses in Table 4.2 must be modified for eccentricity of vertical loading due to its position and seismic moment and the slenderness ratio multiplying factors given in Table 4.3. The effective height h may be taken as a factor times the actual height of wall between floors, the factor being 0.75 when floors are rigid diaphragms and 1.00 for flexible roofs; it will be 2.0 for parapets.

The effective length L will be a fraction of actual length between lateral supports, the factor being 0.8 for wall continuous

Table 4.2 Typical permissible stresses

Mortar mix or equivalent			Permissible stresses		Compression for strength of unit, MPa			
cement	lime	sand	tension MPa	shear MPa	3.5	7.0	10.5	14.0
1	-	6	0.05	0.08	0.35	0.55	0.85	1.00
1	1	6	0.13	0.20	0.35	0.70	1.00	1.10
1	-	3	0.13	0.20	0.35	0.70	1.05	1.25

with cross walls or buttresses at both ends, 1.0 for continuous at one end and supported on the other and 1.5 for continuous at one and free at the other.

4.4 GENERAL CONSTRUCTION ASPECTS

4.4.1 Mortar

Since tensile and shear strength are important for seismic resistance of masonry walls, use of mud or very lean mortars will be unsuitable. A mortar mix cement: sand equal to 1:6 by volume or equivalent in strength should be the minimum. Appropriate mixes for various categories of construction are recommended in *Table 4.4*. Use of a rich mortar in narrow piers between openings will be desirable even if a lean mix is used for walls in general.

4.4.2. Wall enclosure

In load bearing wall construction, the wall thickness ' t ' should not be kept less than 190 mm, wall height not more than $20t$ and wall length between cross-walls not more than $40t$. If longer rooms are required, either the wall thickness is to be increased, or buttresses of full height should be provided at $20t$ or less apart. The minimum dimensions of the buttress shall be as thickness and top width equal to t and bottom width equal to one sixth the wall height.

4.4.3 Openings in walls

Studies carried out on the effect of openings on the strength of walls indicate that they should be small in size and centrally located. The following are the guidelines on the size and position of openings:

Table 4.3 Stress factor for slenderness ratio and eccentricity of loading

Slenderness ratio	Stress factor, K , for eccentricity ratio, e/t							Remarks
	0	0.04	0.10	0.20	0.30	0.33	0.50	
6	1.000	1.000	1.000	0.996	0.984	0.980	0.970	Linear interpolation
8	0.920	0.920	0.920	0.910	0.880	0.870	0.850	may be used.
10	0.840	0.835	0.830	0.810	0.770	0.760	0.730	
12	0.760	0.750	0.740	0.706	0.664	0.650	0.600	
14	0.670	0.660	0.640	0.604	0.556	0.540	0.480	Values for $e/t = 0.5$ are
16	0.580	0.565	0.545	0.500	0.440	0.420	0.350	for interpolation only
18	0.500	0.480	0.450	0.396	0.324	0.300	0.230	
21	0.470	0.448	0.420	0.354	0.276	0.250	0.170	
24	0.440	0.415	0.380	0.310	0.220	0.190	0.110	

Table 4.4 Recommended mortar mixes

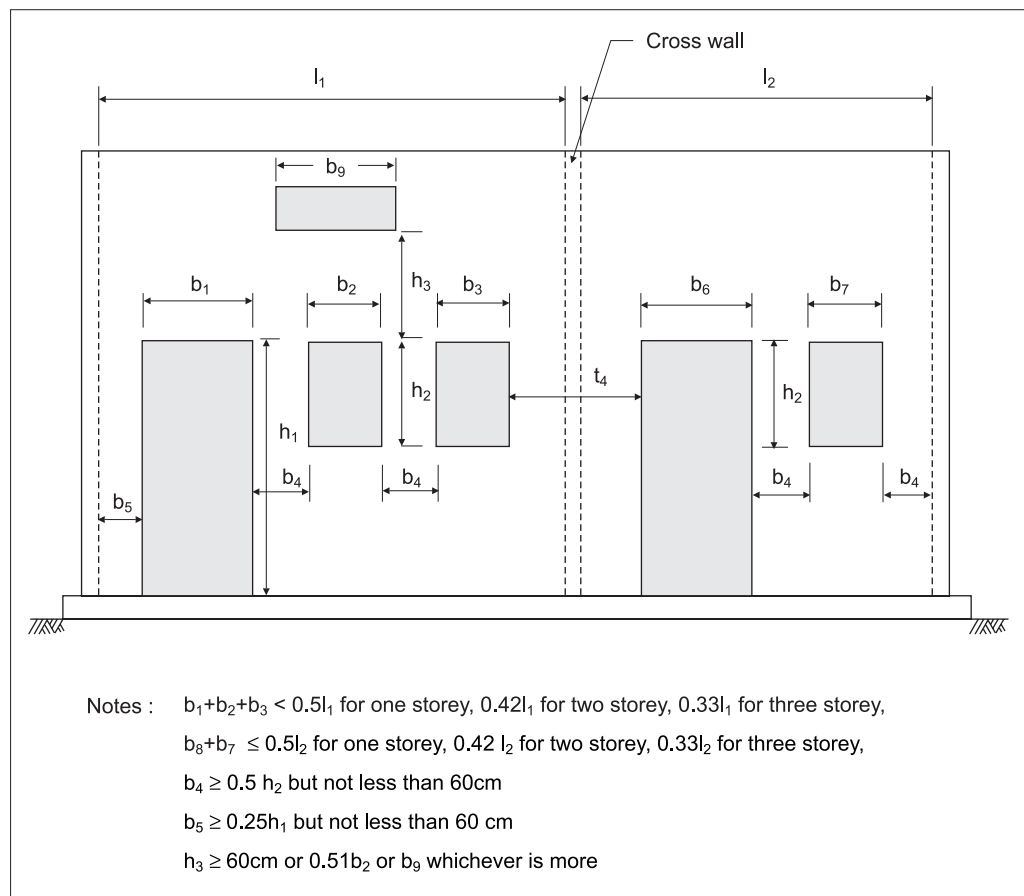
Category of construction*	Proportion of cement-lime-sand
I	Cement-sand 1:4 or cement-lime-sand 1:1:6 or richer
II	Cement-lime-sand 1:2:9 or richer
III	Cement-sand 1:6 or richer
IV	Cement-sand 1:6 or lime-cinder** 1:3 or richer

Notes: * Category of construction is defined in Table 3.1.
 ** In this case some other pozzolonic material like trass (Indonesia) and surkhi (burnt brick fine powder in India) may be used in place of cinder.

- (i) Openings to be located away from the inside corner by a clear distance equal to at least $1/4$ of the height of openings but not less than 60 cm.
- (ii) The total length of openings not to exceed 50 percent of the length of the

wall between consecutive cross walls in single-storey construction, 42 percent in two-storey construction and 33 percent in three storey buildings.

- (iii) The horizontal distance (pier width) between two openings to be not less than half the height of the shorter opening, Fig 4.6, but not less than 60 cm.
- (iv) The vertical distance from an opening to an opening directly above it not to be less than 60 cm nor less than $1/2$ of the width of the smaller opening, Fig 4.6.
- (v) When the openings do not comply with requirements (i) to (iv), they

**Fig 4.6 Recommendation regarding openings in bearing walls**

should either be boxed in reinforced concrete around or reinforcing bars provided at the jambs through the Masonry, Fig 4.7.

4.4.4 Masonry bond

For achieving full strength of masonry the usual bonds specified for masonry should be followed so that the vertical joints are

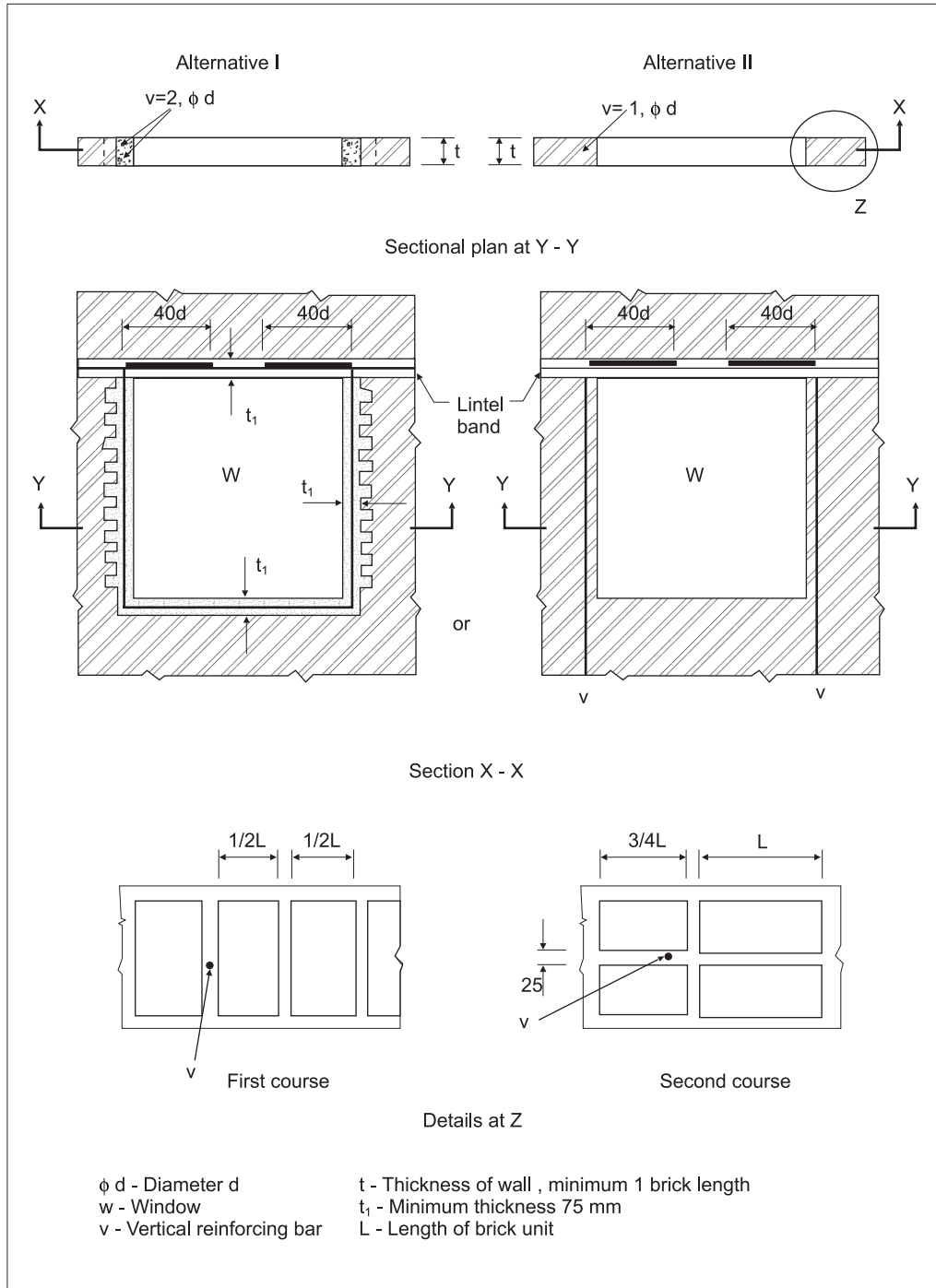


Fig 4.7 Strengthening of masonry around openings

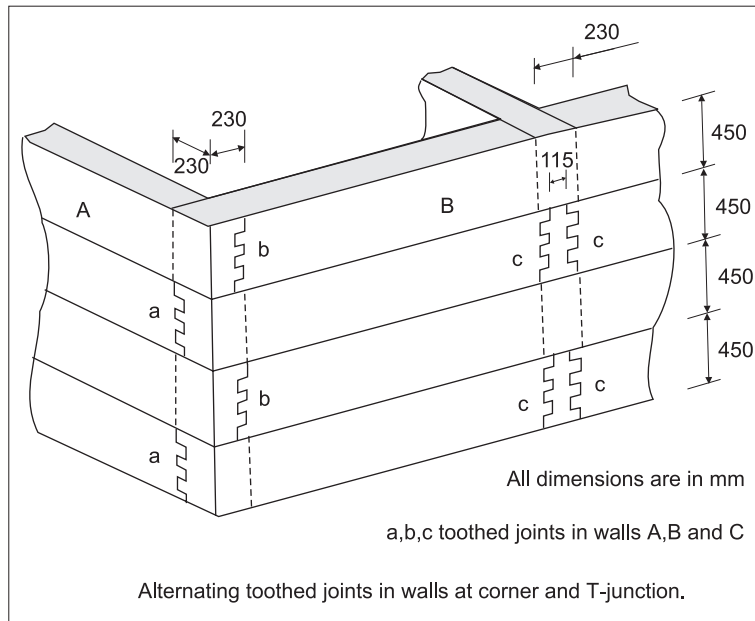


Fig 4.8 A typical detail of masonry

broken properly from course to course. The following deserves special mention.

Vertical joint between perpendicular walls

For convenience of construction, builders prefer to make a toothed joint which is

many times left hollow and weak. To obtain full bond it is necessary to make a sloping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternately in lifts of about 45 cm, Fig 4.8.

4.5 HORIZONTAL REINFORCEMENT IN WALLS

Horizontal reinforcing of walls is required for imparting to them horizontal bending strength against plate action for out of plane inertia load and for tying the perpendicular wall together. In the partition walls, horizontal reinforcement helps preventing shrinkage and temperature cracks. The following reinforcing arrangements are necessary.

4.5.1 Horizontal bands or ring beams

The most important horizontal reinforcing

Table 4.5 Recommendation for steel in RC band

Span, m	Longitudinal steel in R.C. bands							
	category I		category II		category III		category IV	
	no of bars	diameter of bars, mm	no of bars	diameter of bars, mm	no of Bars	diameter of Bars, mm	no of Bars	diameter of Bars, mm
5	2	12	2	10	2	10	2	10
6	2	16	2	12	2	10	2	10
7	2	16	2	16	2	12	2	10
8	4	12	2	16	2	16	2	12
9	4	16	4	12	2	16	2	12

Notes: (i) Width of the RC band is assumed to be the same as the thickness of wall. Wall thickness shall be 20 cm minimum. A cover of 25 mm from face of wall will be maintained. For thicker walls, the quantity of steel need not be increased. For thinner walls, see 4.7.

(ii) The vertical thickness of RC band may be kept minimum 75 mm where two longitudinal bars are specified and 150 mm where four longitudinal bars are specified.

(iii) Concrete mix to be 1:2:4 by volume or having 15 MPa cube crushing strength at 28 days.

(iv) The longitudinal bars shall be held in position by steel links or stirrups 6 mm diameter spaced at 150 mm apart (see Fig 4.10 (a))

(v) Bar diameters are for mild-steel. For high strength must deformed bars, equivalent diameter may be used.

is through reinforced concrete bands provided continuously through all load bearing longitudinal and transverse walls at plinth, lintel, and roof-eave levels, also at top of gables according to requirements as stated hereunder:

- (i) **Plinth band:** This should be provided in those cases where the soil is soft or uneven in their properties as it usually happens in hill tracts. It will also serve as damp proof course. This band is not too critical.
- (ii) **Lintel band:** This is the most important band and will incorporate in itself all door and window lintels the

reinforcement of which should be extra to the lintel band steel. It must be provided in all storeys in buildings as per *Table 4.5*.

- (iii) **Roof band:** This band will be required at eave level of trussed roofs, *Fig 4.9* and also below or in level with such floors, which consist of joists and covering elements so as to properly integrate them at ends and fix into the walls.
- (iv) **Gable band:** Masonry gable ends must have the triangular portion of masonry enclosed in a band, the horizontal part will be continuous with the eave level band on longitudinal

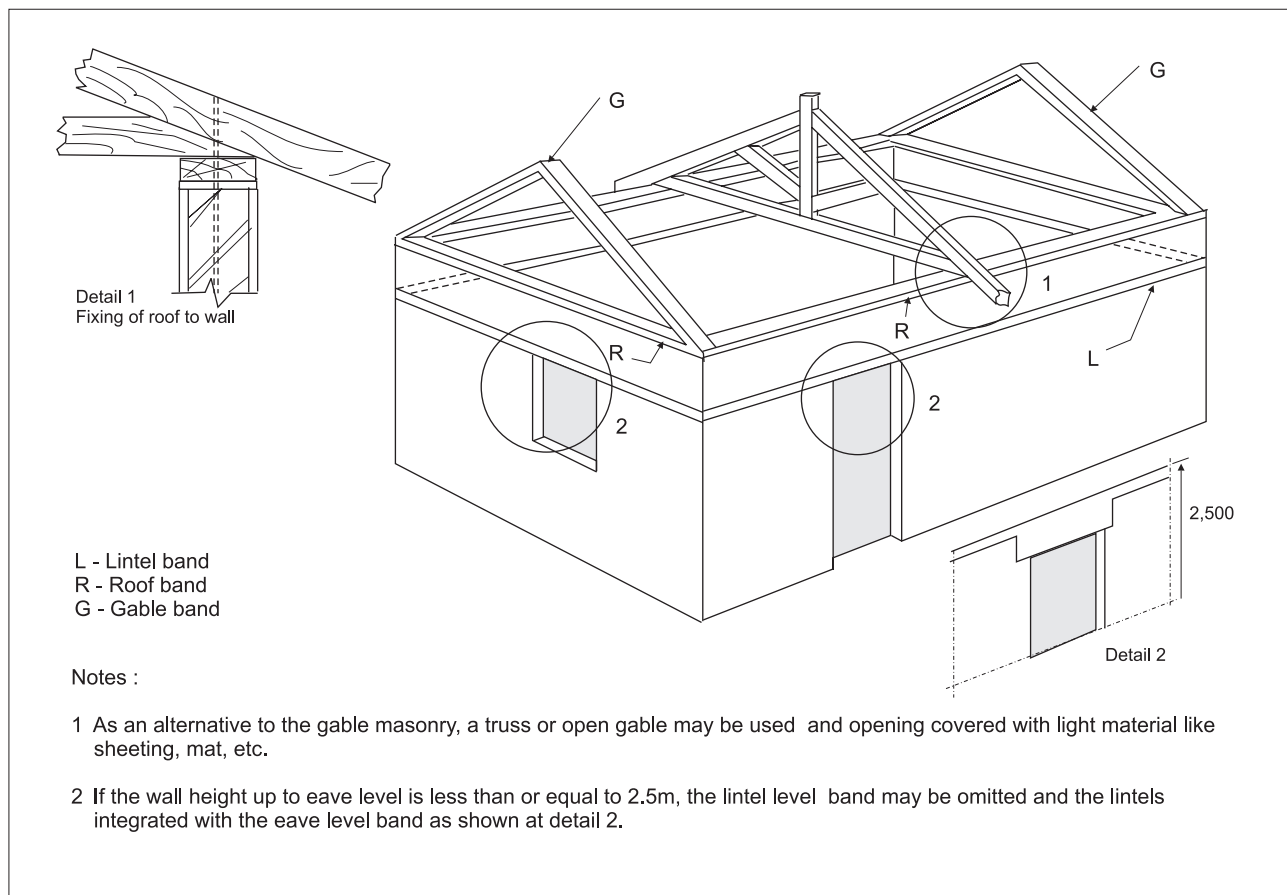


Fig 4.9 Gable band and roof band in barrack type buildings

walls, Fig 4.9.

4.5.2 Section of bands or ring beams

The reinforcement and dimensions of these

bands may be kept as follows for wall spans upto 9 m between the cross walls or buttresses. For longer spans, the size of band must be calculated.

A band consists of two (or four) longitudinal steel bars with links or stirrups embedded in 75 mm (or 50 mm), thick concrete, Fig 4.10. The thickness of band may be made equal to or a multiple of masonry unit and its width should equal the thickness of wall. The steel bars are located close to the wall faces with 25 mm cover and full continuity is provided at corners and junctions. The minimum size of band and amount of reinforcing will depend upon the unsupported length of wall between cross walls and the effective seismic coefficient based on seismic zone, importance of buildings, type of soil and storey of the building.

Appropriate steel and concrete sizes are recommended for various buildings in Table 4.5. Such bands are to be located at critical levels of the building, namely plinth, lintel, roof and gables according to requirements (see 4.5.1).

4.5.3 Dowels at corners and junctions

As a supplement to the bands described in (a) above, steel dowel bars may be used at corners and T-junctions to integrate the box action of walls. Dowels, Fig 4.11, are placed in every fourth course or at about 50 cm intervals and taken into the walls to sufficient length so as to provide the full bond strength. Wooden dowels can also be used instead of steel. However, the dowels do not serve to reinforce the walls in horizontal bending except near the junctions.

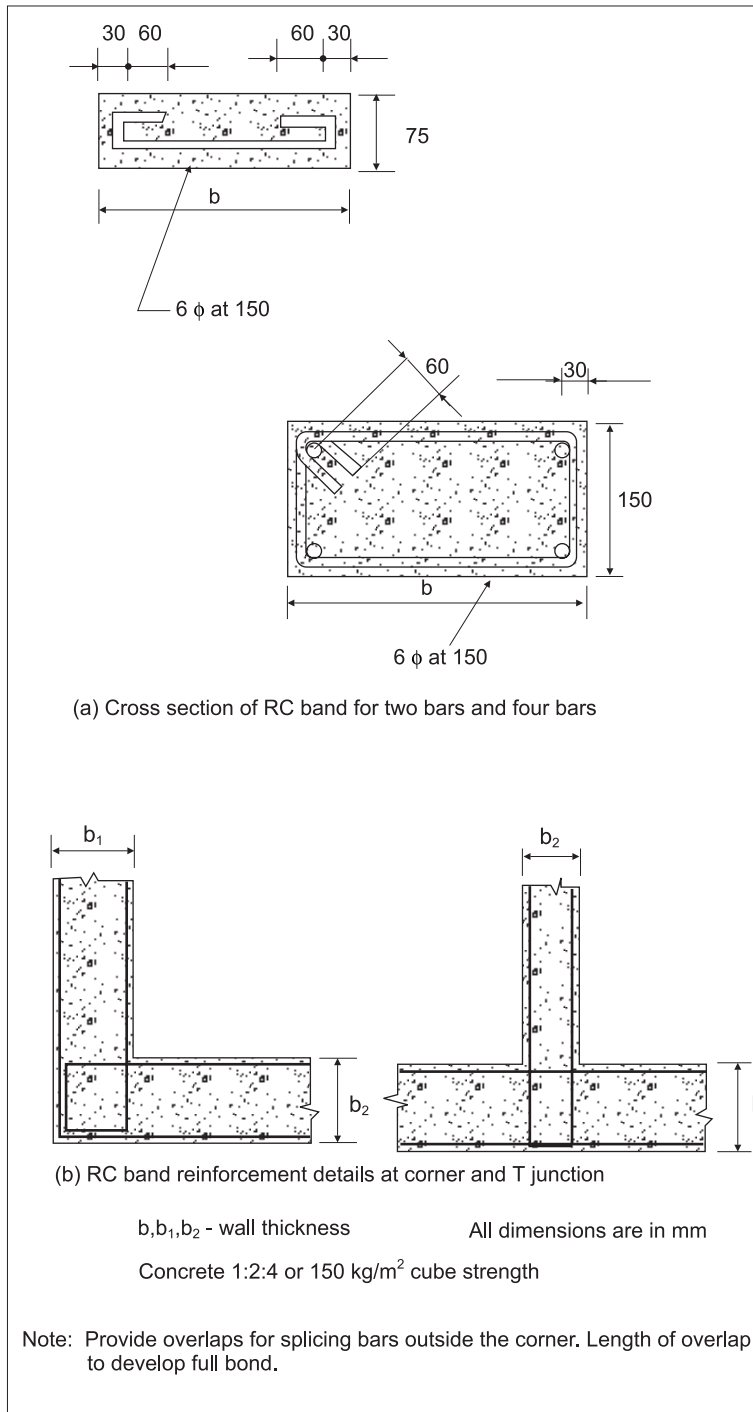


Fig 4.10 Reinforcement in RC band

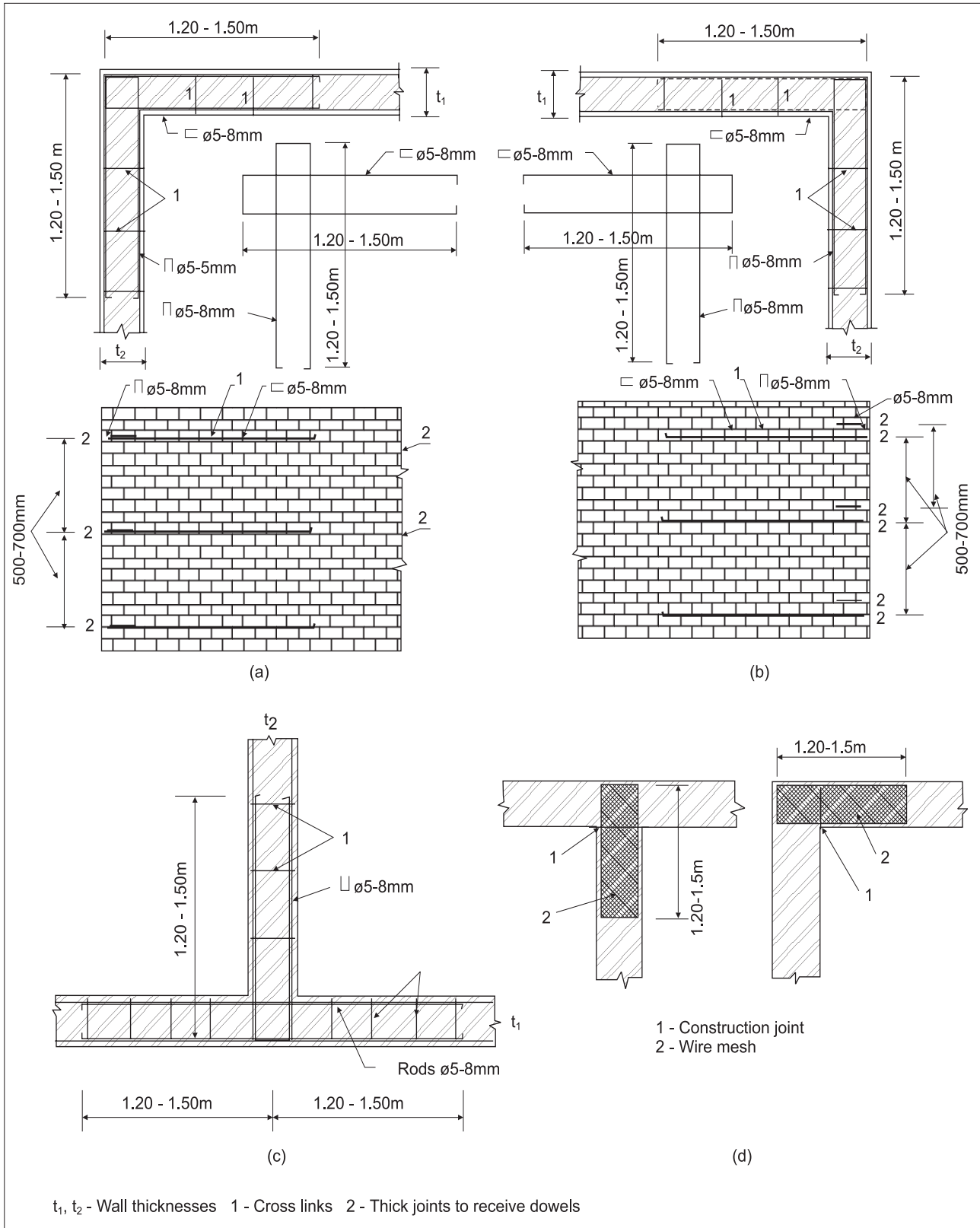


Fig 4.11 (a) Corner-strengthening by dowel reinforcement placed in one joint (b) Corner-strengthening by dowel reinforcement placed in two consecutive joints. (c) T-junction - strengthening by dowel reinforcements (d) Strengthening by wire fabric at junction and corner

Table 4.6 Recommendation for vertical steel at critical sections

No of	Storeys	Diameter of mild steel single bar in mm at each critical section for category (I)			
		category I	category II	category III	category IV
One		16	12	12	Nil
Two	Top	16	12	12	Nil
	Bottom	20	16	16	Nil
Three	Top	16	12	12	Nil
	Middle	20	16	12	Nil
	Bottom	20	16	16	Nil
Four	Top	(2)	(2)	12	12
	Third			12	12
	Second			16	12
	Bottom			16	12

Notes: (i) Category of construction is defined in Table 3.1. Equivalent area of twisted grip bars or a number of mild steel bars could be used but the diameter should not be less than 12 mm.

(ii) Four storeyed load bearing wall construction may not be used for categories I and II buildings.

4.6 VERTICAL REINFORCEMENT IN WALLS

The need for vertical reinforcing of shear walls at critical sections was established in Para 2.6.7. The critical sections were the jambs of openings and the corners of walls. The amount of vertical reinforcing steel will depend upon several factors like the number of storeys, storey heights, the effective seismic coefficient based on seismic zone, importance of building and soil foundation type. Values based on rough estimates for building are given in Table 4.6 for ready use. The steel bars are to be installed at the critical sections, that is the corners of walls and jambs of doors right, from the foundation concrete and covered with cement concrete in cavities made around them during masonry construction. This concrete mix should be kept 1:2:4 by volume or richer. Typical arrangements of placing the vertical steel in brick work are shown in Fig 4.12.

The jamb steel was shown in Fig 4.7. The jamb steel of window openings will be easiest to provide in box form around it. The vertical steel of opening may be stopped by embedding it into the lintel band but the vertical steel at corners and junctions of walls must be taken into the floor and roof slabs or roof band

The total arrangement of providing reinforcing steel in masonry wall construction is schematically shown in Fig 4.13.

4.7 FRAMING OF THIN LOAD BEARING WALLS

If load-bearing walls are made thinner than 200 mm, say 150 mm inclusive of plastering on both sides, reinforced concrete framing columns and collar beams are necessary which are constructed to have full bond with the walls. Columns are to be located at all corners and junctions of walls and at not more than 1.5 m apart but so located as to frame up the doors and windows. The horizontal bands or ring beams are located at all floors, roof as well as lintel levels of the openings. The sequence of construction between walls and columns is: first to build the wall upto 4 to 6 courses height leaving toothed gaps (tooth projection being about 40 mm only) for the columns and second to pour 1:2:4 concrete to fill the columns against the walls using wood-forms only or two sides. Needless to say that column steel should be accurately held in position all along. The band concrete should be cast on the wall masonry directly so as to develop full bond with it.

Such construction may be limited to only two storeys maximum in view of its vertical load carrying capacity. The horizontal length of walls between cross walls may be

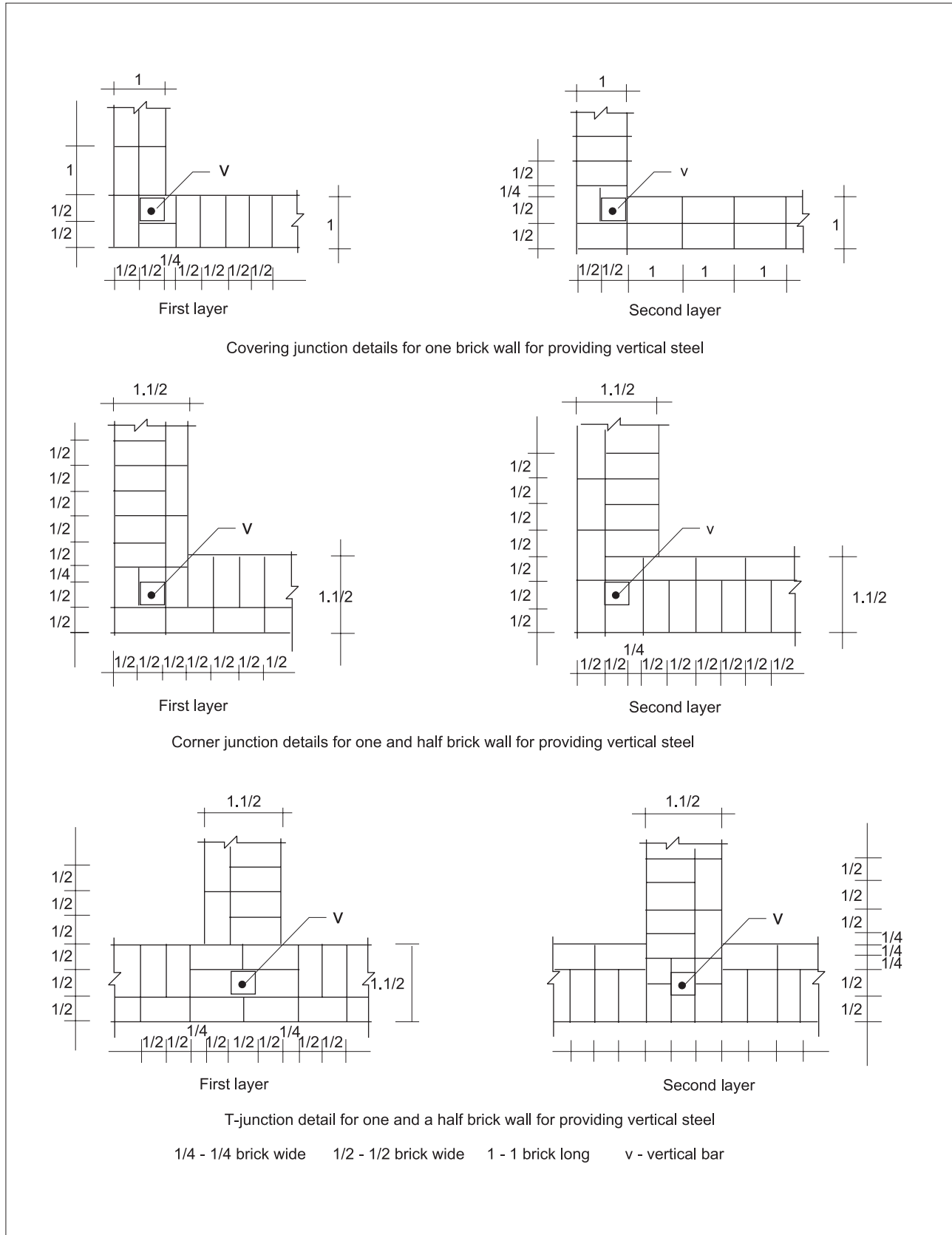


Fig 4.12 Vertical reinforcement in walls

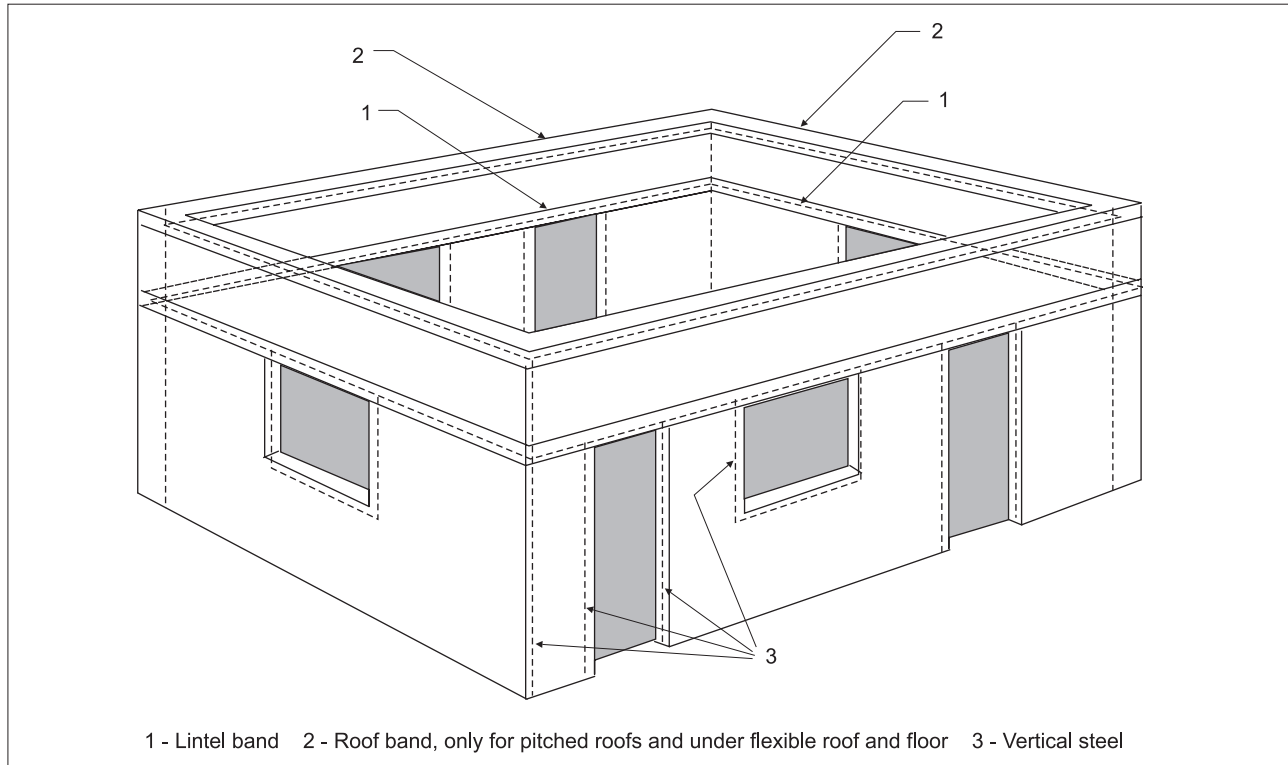


Fig 4.13 Overall arrangement of reinforcing masonry buildings

restricted to 7 m and the storey height to 3 m.

4.8 REINFORCING DETAILS FOR HOLLOW BLOCK MASONRY

The following details may be followed in placing the horizontal and vertical steel in

hollow block masonry using cement-sand or cement concrete blocks.

4.8.1 Horizontal band

U-shaped blocks may best be used for construction the horizontal bands at various levels of the storeys as per seismic requirements, as shown in, Fig 4.14.

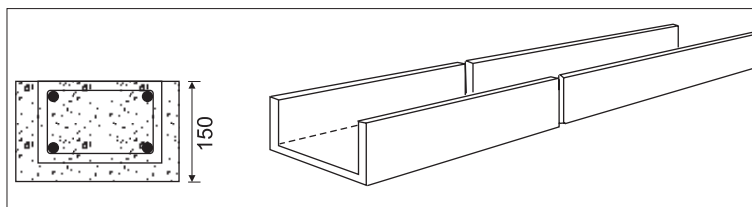


Fig 4.14 U-blocks for horizontal bands

The amount of horizontal reinforcement may be taken 25 percent more than that given in Table 4.5 and provided by using four bars and 6mm dia stirrups. Other continuity details shall be followed as shown in Fig 4.10.

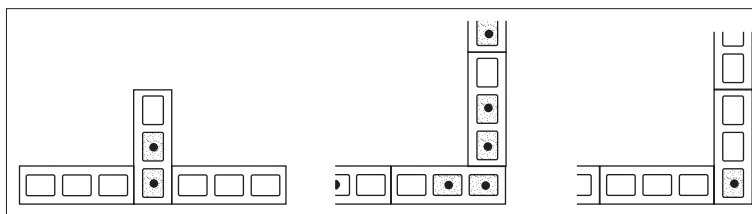


Fig 4.15 Vertical reinforcement in cavities

4.8.2 Vertical reinforcement

The vertical bars as specified in Table 4.6 may conveniently be located inside the cavities of the hollow blocks, one bar in one cavity. Where more than one bar is planned,

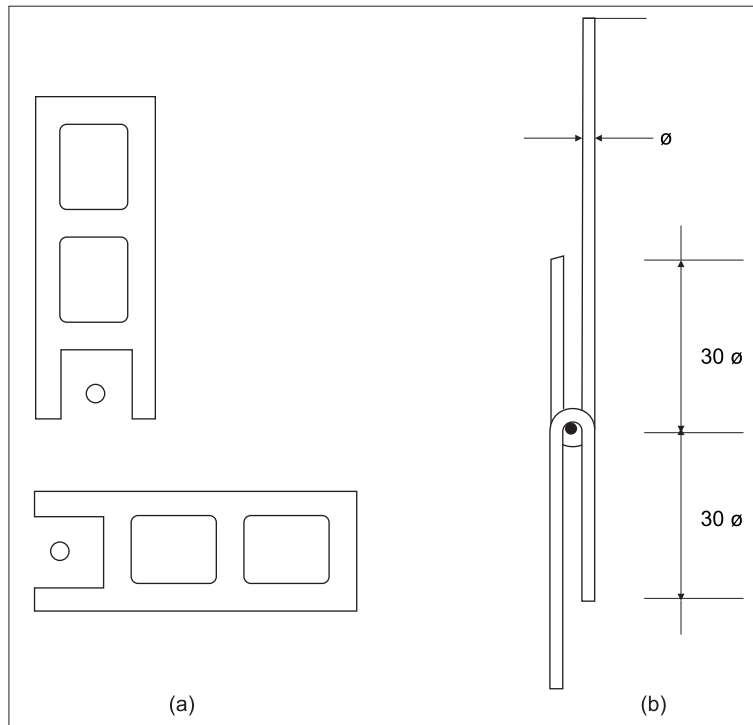


Fig 4.16 Vertical reinforcement in cavities

these can be located in two or three consecutive cavities as shown in Fig 4.15. The cavities containing bars are to be filled by using micro-concrete 1:2:3 or cement-coarse sand mortar 1:3 and properly rodded for compaction.

Practical difficulty is faced in threading the bars through the hollow blocks since the bars have to be set in footings and have to be kept standing vertically while lifting the blocks whole storey heights, threading the bar into the cavity and lowering it down to the bedding level. To avoid lifting of blocks too high, the bars are made shorter and overlapped with upper portions of bars. This is wasteful of steel as well as the bond strength in small cavities remains doubtful. For solving this problem, two alternatives may be used as shown in Fig 4.16 (a) use of three sided or U-block (b) bent interlocked bars.

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